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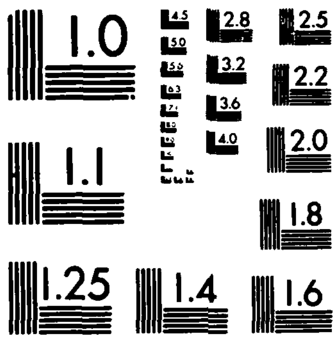
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DESIGN & INSTALLATION OF NEARSHORE OCEAN CABLE PROTECTION SYSTEMS

FPO-1-78(3)

Prepared by

CIVIL ENGINEERING LABORATORY
Port Hueneme, California 93043

for

OCEAN ENGINEERING AND CONSTRUCTION PROJECT OFFICE
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repair operations have been required forsome cables that have been damaged by abrasion, corrosion, and overstressing of the strength members. A cable failure analysis coducted by Cullison (1975) documented the most common causes and occurences for various types of cable failures. In most of the cases, regardless of the type of failure, the overriding reason that the failures occurred could be attributed to either an inadequate understanding of the environment or the improper application of the cable protection system for the type of environmental hazards occurring at the site.

A considerable amount of work has been done to develop techniques, tools, and procedures for the installation, maintenance, and repair of ocean cable systems. Most of these efforts, however, have been confined to a specific type of cable protection system or a very limited range of environmental conditions. Consequently, many cables have been installed through a seat-of-the-pants approach that consists of an arbitrary selection of a protection system and a past experience design philosophy (i.e., "has this system worked in the past"). All too often, very little attempt has been made to access or correlate environmental factors or the economics of using alternative system designs.

Under the sponsorship of the Chesapeake Division of the Naval Fascilities Engineering Command, the Civil Engineering Laboratory has prepared this cable protection handbook in an attempt to alleviate many of the previously encountered problems. Engineering guidelines were developed for use in the selection and design of optimum cable protection systems.

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Chapter 1

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Chapter 1

INTRODUCTION

1.1 BACKGROUND

The U.S. Navy currently has a requirement to install and operate ocean cables for the transmission of power and data. Many of the previous installations have encountered significant problems with reliability and survivability of the cable for the required life of the system. Costly maintenance and repair operations have been required for some cables that have been damaged by abrasion, corrosion, and overstressing of the strength members. A cable failure analysis conducted by Cullison (1975) documented the most common causes and occurrences for various types of cable failures. In most of the cases, regardless of the type of failure, the overriding reason that the failures occurred could be attributed to either an inadequate understanding of the environment or the improper application of the cable protection system for the type of environmental hazards occurring at the site.

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Under the sponsorship of the Chesapeake Division of the Naval Facilities Engineering Command, the Civil Engineering Laboratory has prepared this cable protection handbook in an attempt to alleviate many of the previously encountered problems. Engineering guidelines were developed for use in the selection and design of optimum cable protection systems.

Four basic objectives were established to meet this goal. These objectives included:

(1) Identify the parameters that influence the selection, design, and installation of various cable protection systems.

(2) Document the systems, techniques, and methods used previously (or proposed for future work) to protect ocean cables, and present currently available data on each of these systems that would influence the selection and design process.

(3) Develop a theory that would allow a system to be designed based on engineering principles, and incorporate the effects of relevant environmental and system parameters.

(4) Present a design methodology that would integrate the information presented in this handbook and allow for the rational selection and specification (design) of a cable protection system.

1.2 SCOPE

This handbook deals with the protection of that portion of an ocean cable system which passes through the nearshore zone. This zone is defined as an indefinite area that extends seaward from the shoreline to well beyond the breaker zone. For the purpose of this work, the outer limit of this zone has been established as the distance from shore at which the water depth is great enough that the hydrodynamic effects of storm waves no longer represent a potential danger to the bottom-resting cable. Although this gives a rationale for establishing the nearshore zone, the extent of this region depends on specific site conditions. It could extend offshore to a water depth of at least 100 feet or as much as 600 feet (Valent and Brackett, 1976). Figure 1-1 shows the relationship of the nearshore zone to other coastal regions.

The information presented in this handbook is directed primarily toward future cable installations; however, many of the protection systems and most of the design theory can easily be adapted for use in repair of existing installations.

A Cable Protection System is any hardware, equipment, procedure, or combination thereof that allows an ocean cable to survive potential hazardous environmental conditions during the required operational life of the system. The cable protection system differs from the individual techniques discussed in Chapter 4 in that the system may consist of a combination of several techniques. The various cable protection techniques have been separated into three basic categories that depend upon the type of protection and the manner in which the design theory is applied. These three basic categories are: (1) stabilization, (2) im-

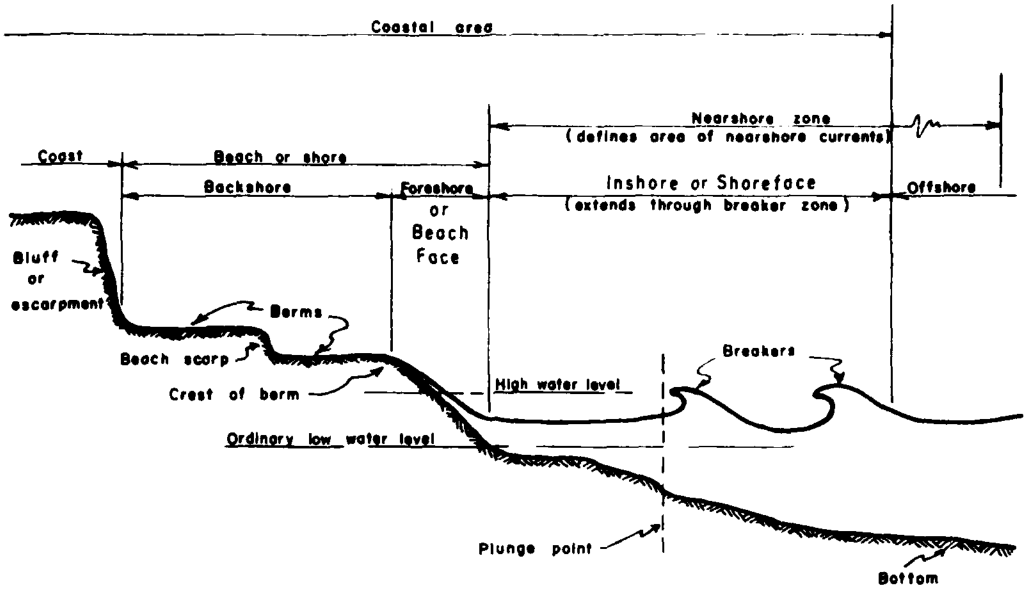

Figure 1-1. Relationship of nearshore zone to other coastal zones.

mobilization, and (3) burial. The technique of tensioning is discussed separately, because it does not fit any of the definitions established for the three major categories.

- Stabilization Techniques - Any cable protection technique that, by virtue of its weight and the resulting friction between the seafloor and the system components, allows the cable to resist environmental hazards. A mass anchor can be considered a protection system if it succeeds in resisting the environmental influences (hydrodynamic, anchor drag forces, etc.). However, when the environmental influences exceed the friction forces, the mass anchor becomes part of the cable system and may itself require additional protection.

- Immobilization Techniques - Any cable protection technique that mechanically couples the cable to the seafloor at discrete points along the length of the cable. Immobilization greatly reduces movement but, theoretically, can not totally eliminate all motion of an unstable cable. The majority of the design theory section deals with this type of technique.

• Burial Techniques - Any technique that provides protection by allowing the cable to be placed below the surface of the seafloor. The effectiveness of these techniques depends on their ability to remove the cable from an environment that may be hazardous. Since burial eliminates the influence of the environmental hazards rather than providing a means to resist them, the design theories presented are not applicable. The selection and implementation of one of these techniques depends, therefore, on economics and the ability of the selected equipment to bury the cable to the required depth (to avoid the potential hazard).

The individual techniques that are discussed under each of the basic categories include:

(1) Mass anchors (stabilization)
  (a) Armor wire
  (b) Split-pipe
  (c) Pipe casing
  (d) Concrete
  (e) Chain

(2) Tie-downs (immobilization)
  (a) Pins
  (b) Grouted fasteners
  (c) Rockbolts

(3) Burial
  (a) Self-burial
  (b) Jetting
  (c) Dredging
  (d) Explosive excavation
  (e) Mechanical trenching
  (f) Drilled hole

Although tensioning has rarely been used as the only means of protecting an ocean cable, it is presented as a separate technique since it can be used with almost any of the other cable protection techniques to reduce the magnitude of the displacement produced by hydrodynamic forces. The effects of tensioning on the system design are discussed in detail in Chapter 6.

1.3 PRESENTATION OF MATERIAL

This handbook has been prepared with the objective of bringing together in one document all the necessary information, data, and engineering analysis procedures required for the ocean engineer to select,

design, and plan the installation of a cable protection system. The material has been arranged in the order in which it would be encountered in an actual system design.

The first step in any design process for a cable protection system is to assess the conditions and potential hazards that will be encountered at a particular site or sites of interest. Chapter 2 discusses 22 parameters that affect the selection, design, and/or installation of a cable protection system. This chapter is intended as an aid in setting up a site survey plan so that all relevant site characteristics and mission requirements will be investigated and that the data obtained from the survey will be formatted such that it can be used throughout the remainder of the design and installation planning phases.

Chapter 3 discusses the effect each parameter has on various cable protection techniques. Since not all of the protection techniques are technically feasible at any one site, this chapter also provides a preliminary procedure for screening the protection techniques based on the results of the site survey analysis. Seven of the 22 parameters discussed in Chapter 2 were found to have a significant effect on the feasibility of using each of the cable protection techniques. For each of these seven parameters a feasibility assessment matrix is presented that provides an indication of the applicability of each technique for various possible ranges of the environmental parameter being considered. At the conclusion of this screening process a list of techniques that are compatible with the environmental conditions at the site is obtained.

Chapter 4 is a discussion of the individual cable protection techniques. This discussion includes a description of major components and previously used or proposed installation techniques, estimates on manpower requirements and production rates, and when available any data that relate to the design or installation of the system. Also included is a more detailed evaluation of the technique feasibility for each of the selection parameters; it is presented in narrative rather than tabular form. This section is intended to provide a cross check for the preliminary screening process and to allow a more detailed assessment where some doubt to the technique applicability may exist. The preliminary screening list obtained from Chapter 3 is intended to guide one to the applicable sections of this chapter.

Chapter 5 is a review of wave kinematics and hydrodynamic force analysis as they relate to the design of ocean cable protection systems. This chapter provides the necessary equations, tables, and graphs to carry out the required wave and current loading analysis without reference to other texts. This information has been included in the hope that it will be useful if on-site design changes are required and an extensive reference library is not available.

The design theory for both stabilization and immobilization of ocean cables is presented in Chapter 6. The stabilization system design is based on the equilibrium of hydrodynamic and friction forces, while the immobilization system theory is developed from a strength of materials analysis of the system components and the equilibrium of internal and external forces.

Chapter 7 summarizes the material presented in the previous five chapters and establishes a systematic procedure for analyzing the hydrodynamic loads on an ocean cable and, subsequently, designing a stabilization and/or immobilization system that allows the cable to resist these loads. This chapter also contains an outline of factors that influence the economic feasibility of installing cable protection systems.

Chapter 2

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Chapter 2

SITE SURVEY

2.1 INTRODUCTION

This chapter is intended to serve as an aid in selecting suitable cable landing sites and then in setting up a survey of potential sites. A general description of the seafloor close to potential cable landing points can be obtained from marine charts and local fishermen, boaters, nearshore residents, etc. This information, along with other background data on currents and winds, provides a preliminary model of the beach and nearshore from which potential cable landing routes can be laid out. Following this preliminary selection a detailed survey along all potential cable routes will be required in order to provide the input necessary to select the best route.

There are a few major parameters that must be taken into account when making a decision on preliminary route selections. For instance, areas of exposed rock should be avoided because the cost of immobilizing and protecting cable on rock is higher than that on soil. Furthermore, even the type of soil and inclusions, if any, can be very important: on sands, wave action will cause a weighted cable to sink below the seafloor surface, provided the cable is not hung-up on rock inclusions or cohesive soil layers. On clays, cables may sink if the soil is very soft, but wave action will not play a part in the sinkage mechanism. Also, areas of extreme topographic change should be avoided. Often cable routes across the nearshore are set in natural troughs or ravines to protect the cable from lateral currents. The wave and current-generated forces on the cable dictate the cable weighting and/or tie-downs necessary to immobilize the cable. Waves and currents also influence the effectiveness of divers surveying the cable route, installing the cable system, and then immobilizing and protecting the system. Thus, in order to minimize cost, if all other factors are equal, areas with lower waves and currents should provide the best cable landing route. The availability of logistical support, or the accessibility of that support to the beach, can also significantly influence route selection.

Twenty-two parameters have been identified that affect the selection, design, and/or installation of a cable protection system. These parameters are presented so that all relevant site characteristics and mission-related requirements will be investigated and that the data

obtained will be formatted such that it can be used throughout the remainder of the design and installation planning phases. The discussion of these parameters is divided into four major categories: (1) Environmental Parameters, (2) Hazards, (3) Operational Support Requirements, and (4) Mission-Related Requirements.

2.2 ENVIRONMENTAL PARAMETERS

Environmental parameters address those characteristics of the operating environment that have a direct effect on the use or performance of the cable system and in most cases determine the feasibility/adequacy of the various stabilization systems. These parameters are specific to each individual site and must be known (either through prediction or direct measurement) to assure effective operation of the system. These parameters include (1) bottom material; (2) topography; (3) water visibility; (4) water depth; (5) chemical and biological characteristics; and (6) waves, current, and wind.

2.2.1 Bottom Material Characteristics

Natural earth materials are generally categorized as soil or rock. In reality there is no sharp distinction between soil and rock; rather, there are a few arbitrary dividing lines selected by various groups to best serve each group's special interests.

Definition/Classification. *Soil is a natural aggregate of mineral grains and animal shells, with or without organic constituents, that can be separated by gentle mechanical means, such as agitation in water (adapted from Peck et al., 1953).* Soils can be classified best for engineering purposes by the Unified Soil Classification System (Interior 1968; NAVFAC, 1971).

Rock must be defined both in terms of its individual structural units, such as a monolithic block in a jointed rock mass, or in terms of the massive unit, such as a 200-m cube of rock. Rock, the individual structural unit, is a natural aggregate of mineral grains connected by strong and permanent cohesive forces (Peck et al., 1953). The rock mass may often be considered an aggregate of separate rock particles or blocks, whose relative cohesion depends on the intensity and frequency of foliation and jointing in the rock mass (Farmer, 1968).

Rocks are classified first genetically based on their origin, and then petrographically based on their texture and mineralogy. Classification includes an appraisal of the degree of chemical alteration (Travis, 1955). Classification of a rock mass for engineering design, including appraisal of the economics of trenching by machine or by blasting or appraisal of the serviceability of various rockbolts as cable anchors, must include information on foliation and joint direction, spacing and inclination, and the position and inclination of any faults (Farmer, 1968).

Data Requirements. The first data sources to be examined are available charts of seafloor material and local experience of fishermen, boaters, nearshore residents, etc. On soil seafloors it is necessary to classify and determine the areal and vertical extent of soil type layers or strata. Soil samples can be obtained with small gravity corers or vibratory corers operating from a work boat and surficial samples can be obtained by divers. Soil samples, even somewhat disturbed samples, can then be used to classify the soil according to the Unified Soil Classification System (Interior, 1968; NAVFAC, 1971). Data required from the samples for this classification may often be limited to a visual textural and consistency description; sometimes a grain size distribution will be necessary; and occasionally quantitative consistency tests, the Atterberg Limit determinations, will be necessary.

Reasonable estimates of the depth of soil strata can be obtained using nearshore subbottom profiling techniques demonstrated by Ciani and Malloy (1975). Alternatively, corers can be used to obtain data on the thickness of soil layers; however, coring will generally prove more expensive and less accurate than acoustical subbottom profiling for cable route evaluation.

Close attention must be paid to seeming anomalies. For example, boulders in the seafloor surface will often indicate near surface rock that would impede cable burial operations and cause a cable to be exposed to wave and current action during a cycle of erosion. In some instances, it is prudent when plowing-in or jetting-in cables to make a trial transverse with the cable burial device (without the cable) to verify site conditions and proper functioning of the burial device.

When preliminary information suggests that shallowly buried or outcropping rock may impact on the selection of a cable route, then available nearshore and terrestrial geologic data in the form of charts, reports, and even unpublished data should be sought to obtain a regional picture of the geologic make-up. When this available data has been assembled and evaluated to provide a preliminary picture of the local geology, then an adequate first-hand site survey can be planned. This site survey should obtain fresh rock samples from outcropping rock and use these: (1) to augment and modify the geologic and rock topography maps where necessary, and (2) to obtain a rough measure of rock compressive strength. Core samples are nice but not necessary for this effort. Data on the type of rock and its mass nature will be sufficient to decide on the suitability of trenching, blasting, drilling, and rock bolting in that particular environment (Ciani et al., 1974). Areas of sediment cover should be mapped and their thickness determined as such pockets may dictate cable route selection. The data required for horizontal drill-hole planning are considerably more detailed, highly site specific, and best obtained and evaluated by those doing the drilling; hence, those data are not described here.

Equipment Available to Gather Data. Soil samples are gathered with conventional short-coring equipment. Subbottom profiles can be conducted using conventional low frequency sonic echo ranging equipment. The necessary soil classification testing equipment is basic to all soils testing laboratories. Required accuracies for such soils parameter values are not generally amenable to specification. The gathering of good, interpretable, rock data is best done by divers obtaining hand samples from outcrops or core samples with portable underwater drills (Brackett et al., 1976). Rock mass data are obtained by visual inspection of the rock surface, with subbottom acoustic profiling equipment often providing supplementary data.

Reporting. Seafloor material data are generally reported as a projected profile of classification and pertinent properties versus elevation along the proposed cable route (Figure 2-1).

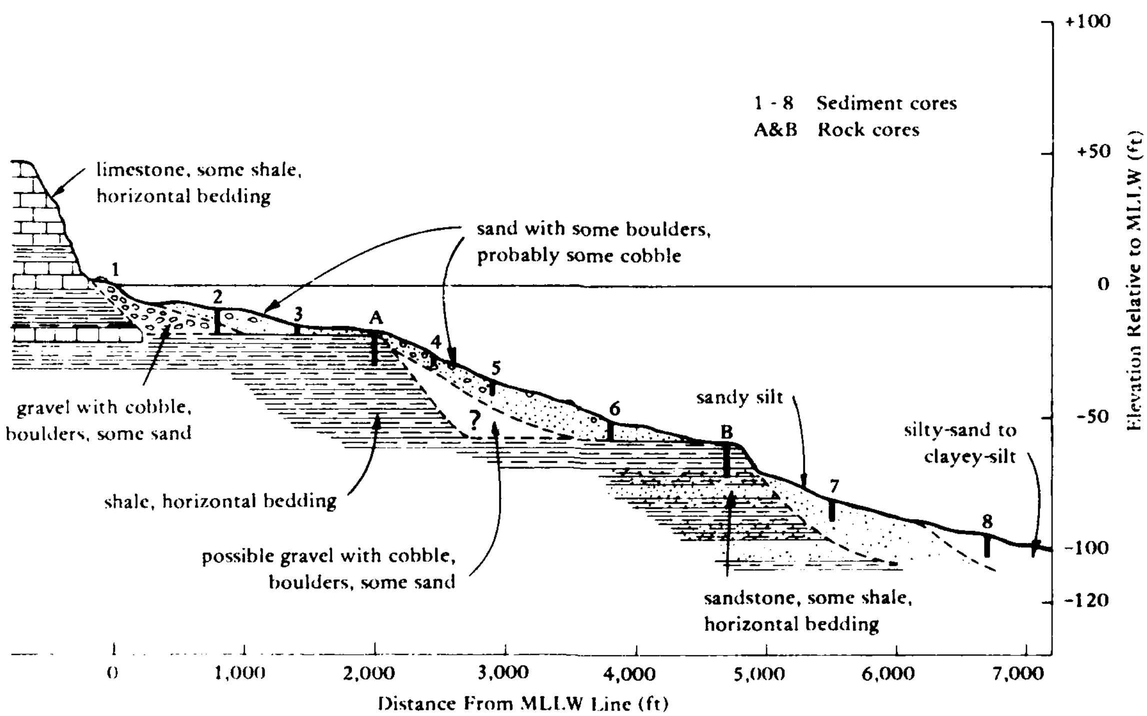


Figure 2-1. Hypothetical cable landing route profile.

2.2.2 Topography

Definition. Topography is defined as the configuration of the surface of the seafloor, including its relief. The three broad categories of topography (and the height of vertical relief that defines these categories) are macrotopography (>60 ft; >20 m),* microtopography (5 to 60 ft; 1.5 to 20 m), and surface roughness (<5 ft; <1.5 m) (NAVFAC P-906, II). The work area for nearshore cable protection/immobilization is usually defined as the area bounded by the mean high water line, the 90-foot (30-m) contour, and the seaward projections of the edges of the on-shore easement. The microtopography of the work area is the most significant category of topography for cable protection/immobilization. The macrotopography may also be of interest (particularly when steep slopes are encountered), but surface roughness over the whole work area is usually not important.

A cable route topographic profile may be defined as a critical section of the surface of the seafloor along the cable route (Ciani, 1974). The surface roughness is much more important along the cable route than it is over the whole work area, particularly in shallow water where the surface tends to be rougher. The importance of microtopography and less so the macrotopography of the cable route is the same as it was for all of the work area.

The surface roughness along the cable route is particularly significant for cable protection/immobilization. The existence and location of natural (rock outcrops and large stones) or man-made (in-place cables or discarded material) obstacles must be known. These obstacles must be avoided or removed to accommodate cable protection systems. This topographic information is also significant for trenching if this is required.

The water depths and slopes along the cable route are also important pieces of topographic information for cable protection/immobilization systems. Water depths are significant for divers performing work on the seafloor because the amount of bottom time is controlled by the water depth. Slopes are significant for the design of such systems because steeper slopes may require additional protection in the form of wire armor or split pipe. Slope is also an important consideration in cable laying because cable payout and ship speed must be coordinated differently for descent laying, when the ship is moving in the downslope direction, or ascent laying, when the ship is moving in the upslope direction (NAVFAC P-906, Vol. II, p. 210).

Data Requirements. The data required for the description of topography are the depths and locations of points in the work area and along the cable route. These depths are referenced to some elevation datum, which is usually locally defined in terms of some mean sea level.

*Since these values are not absolute, their metric equivalents have been rounded off for convenience.

The locations are usually determined relative to reference points in the area, such as horizontal control geodetic survey points, visible landmarks, or simply temporary marks placed specifically for the cable system operation of which this data gathering is a part.

The accuracies required for the measurement of topography in the work area are not high, except along the cable route, which will be discussed next. The lower limit (5 ft; 1.5 m) of the most significant category of topography in the work area, the microtopography, controls this accuracy requirement. Any feature less than 5 feet (1.5 m) in height would be classified as surface roughness which is not important over the whole work area. Therefore, a 5-foot (1.5-m) accuracy is adequate for the work area.

The accuracy required along the cable route is higher than it is over the whole work area. This is true because surface roughness (features less than 5 feet (1.5 m) high) along the cable route is an important consideration. Optimally, a 1-foot (0.3-m) accuracy is desired along the cable route. Lower orders of accuracy may be adequate, depending on the planned installation and the required cable protection/immobilization systems, but the accuracy should never be less than 5 feet (1.5 m), the upper limit of the surface roughness category.

Equipment Available to Gather Data. Various types of equipment are available for topographic depth measurements, including contact sounders (e.g., lead lines), sonic instruments (e.g., fathometers), and laser sounders. These are all described by Ciani (1974). Echo sounders (fathometers) are usually employed for topographic surveys for cable system installations because they are rapid, easy to use, and not difficult to interpret.

Surface positioning methods can be put in two categories: (1) methods that allow a continuous monitoring of position, called analog, and (2) those that identify the position at discrete points, called digital. Analog systems include radionavigation and dead-reckoning; digital systems include satellite navigation and horizontal sextant angle fixes. Nearshore hydrographic surveyors prefer analog positioning systems and analog depth measurement (echo sounding). This is because analog methods yield far more information from which complete maps can be made, and there is less likelihood that significant seafloor features will be missed. Often it is necessary in hydrographic surveys in the surf-zone to use digital positioning methods to augment hydrographic data. Digital positioning techniques are particularly useful in beach profile measurement, because they are cheaper and easier when surf conditions are not severe.

The most accurate analog positioning method in the nearshore zone is the ship-based microwave multiple ranging system using shore-based transceivers. The primary example of this type of analog positioning is Cubic Autotape Model DM-40, which is claimed to be accurate to ±2 feet (±0.7 m) plus 0.001% of range. The most accurate ±1 foot (±0.3 m) digital positioning technique that can be operated from sea is the one involving horizontal sextant angle fixes. Transit triangulation from

shore is a useful digital positioning technique that is even more accurate than horizontal sextant angle fixes, but it involves far more men, pieces of surveying equipment, and time.

Another technique, which is digital but can be used with analog depth measurement equipment, is to keep the survey boat on a line by aligning two objects of fixed position and to make periodic bearing or distance measurements. The periodic bearing measurements can be made with sextant or transit.

For cable route surveys the surface-positioning technique should be analog if possible and digital as backup. The sextant and more so the transit optical techniques are limited by surface visibility, but microwave positioning is not. The microwave equipment is, of course, more expensive and requires the accurate placement of transceivers on shore by trained personnel (Ciani, 1974).

Other equipment options available for topographic surveys at sea are described in NAVFAC P-906 (Vol. II, pp 184-194) and in the Criteria and Methods Reports referenced by Ciani (1974).

Reporting. The format used for reporting topographic data is normally graphical. The topography of the work area is reported with a map showing the cable route, points of significant features, and contours of depth (Figure 2-2). The topographic information along the cable route is reported as a profile showing the depth as a function of distance (Figure 2-3).

2.2.3 Underwater Visibility

Definition. Underwater visibility is defined as the mean greatest distance prevailing over which a large object (cable, boulder, etc.) can be seen and identified. Two points of reference are important to the definition of this parameter. One measure of underwater visibility is the distance or depth into the water to which objects can be seen and identified from the surface. The other is the maximum distance a diver can see through the water column and identify an object. Different factors affect underwater visibility when viewed from these two reference points.

Visibility of submerged objects from the surface is affected by turbidity, surface water roughness, reflection of the sun off of the water, and contrast between the object and the surroundings. Underwater visibility is affected by turbidity, water depth, contrast between the object and surroundings, and size of the object. Turbidity is the most important factor in limiting visibility.

Data Requirements. Underwater visibility is often a seasonal condition that is affected by increased growth of marine organisms (plankton, etc.), run off after storms, and heavy surf. It is, therefore, often best to interview local divers about seasonal conditions if the site survey is conducted more than a couple of months in advance of the actual stabilization operation. If there is no local knowledge of the variation of this parameter, measurements should be made during the site survey.

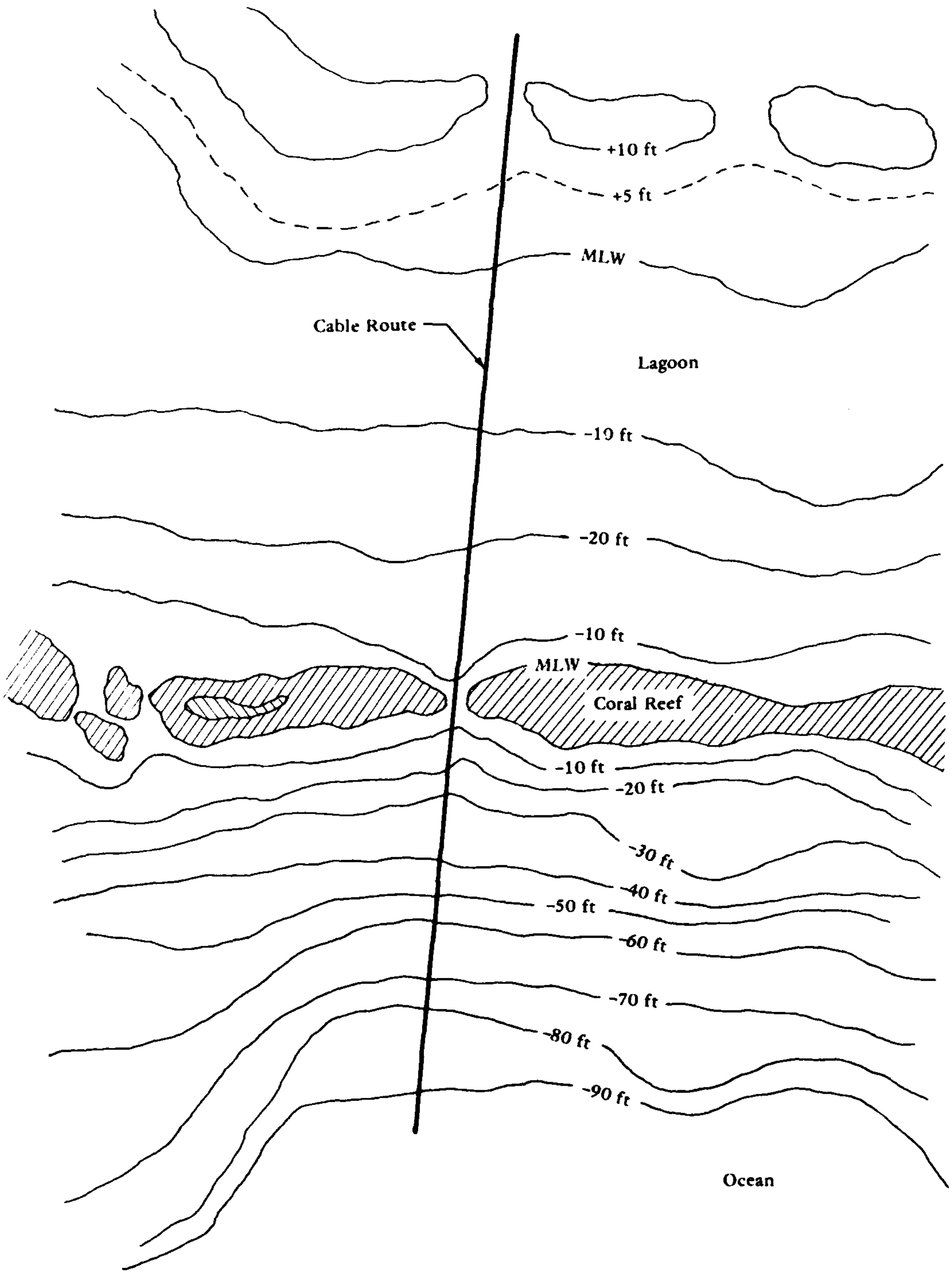


Figure 2-2. Typical topographic data display of work area.

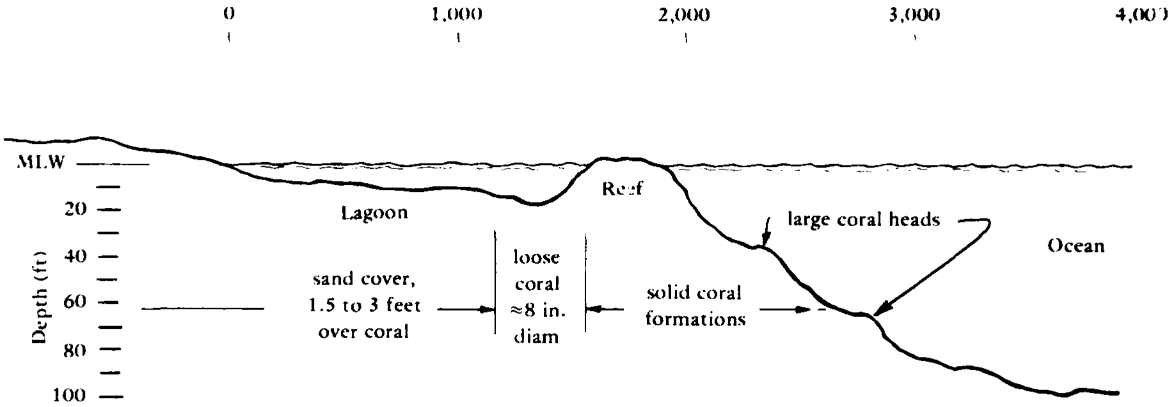


Figure 2-3. Typical topographic data display along cable route.

Equipment Available to Gather Data. Numerous techniques are available for obtaining the data necessary to define the underwater visibility at a given site. The most common techniques include: (1) transmissiometers, (2) direct measurement, and (3) estimation. Logistics and time constraints will govern which of these techniques is utilized.

Transmissiometers are commercially available from a number of companies that supply oceanographic equipment. Table 2-1 lists some of the characteristics of these instruments.

Table 2-1. Commercially Available Transmissiometers

Manufacturer	Model No.	Power Requirements	Range and Accuracy
Interocean Systems, Inc.	500 CSTD	12 VDC	0-200 JTU,[a] ±3%
Hydroproducts Dillingham Corp.	612S	–	0-100%, ±3% at 75% trans.
MARTEK Instruments, Inc.	XMS In-situ	12 VDC or 105-125 VAC	0.1-2.6 meters^{-1} for 1 meter path length, ±1.5%

[a]Jackson Turbidity Units.

Additional instruments and equipment for measuring water clarity are discussed in U.S. Navy Hydrographic Office Special Report No. 41 (1960).

Most of the transmissiometers come as part of a sophisticated oceanographic instrumentation system that can also monitor salinity, temperature, pH, depth, etc.; consequently, these systems are expensive.

Direct measurement can be accomplished by two divers using a distance line or tape measure. They separate to a distance where they can just distinguish each other and then measure this distance. This should be done in several locations along the cable route, starting immediately seaward of the surfzone and at several locations out to the end of stabilization area.

Where visibility exceeds 50 feet (15 m), it is often more convenient to determine the visibility from the surface. One observes the seafloor through a plexiglass view box as the surface support boat moves seaward; the water depth (visibility) at the point where the seafloor begins to fade can be determined either by a lead line or fathometer.

The last and least desirable technique is to estimate the visibility based on the subjective judgment of the diver. Past experience has shown the reliability of this technique to be poor, except in very limited visibility (less than 6 feet; 2 m) where the diver can reference the visibility to his arm length or height.

The desired accuracy of water visibility data is 1 foot (0.3 m) (Ciani et al., 1974); however, visibility greater than 30 feet (10 m) has very little impact on selection of the immobilization technique, and an accuracy of ±01% seems adequate for visibility greater than this.

Reporting. Visibility data should be reported in tabular format, showing the average visibility for each month of the year during which the stabilization operation may take place. If a wide variation in visibility is noted along the proposed cable path, this condition should be reported as a function of water depth or distance from shore.

Related Parameters. Increased wave height will as a rule decrease visibility in the nearshore region, depending on bottom composition. If the site survey is conducted during a relatively calm period, the projected wave and surf conditions (Section 2.2.6) during the stabilization operation should be reviewed for possible impact.

Several of the stabilization techniques discussed in Chapter 4 create poor visibility, such as rock drilling, jetting, and trenching (both mechanical and blasting). Long-shore currents (Section 2.2.6) can be beneficial in clearing the suspended particles from the work site. In calm, protected waters, the turbidity caused by these techniques and their equipment can rapidly reduce visibility to the point where diving operations become difficult.

2.2.4 Water Depth

Definition. Water depth is the vertical distance from the sea surface to the seafloor. In some areas of the world, large tidal variations can cause substantial change in the water depth at a specified location during a 24-hour period.

Data Requirements. Data relating water depth to distance from shore along the proposed or actual cable route are required. The methods for gathering these data are discussed in Section 2.2.2; in addition, data on tidal variations during the period of the stabilization operation should be included.

Equipment Available to Gather Data. See Section 2.2.2.

Reporting. These data are usually presented in chart form, showing the water depth as a function of horizontal distance (seafloor profile). For planning purposes, it is also useful to present in tabular form the horizontal distance between each 10-foot depth contour. This information will allow one to determine the number of divers (if required) needed to support the operation without causing undue delays.

Related Parameters. Water depth is closely related to seafloor topography. When the topographic relief is referenced to a known seafloor datum (usually mean low water), all of the data required to identify water depth are available.

2.2.5 Chemical and Biological Characteristics

Definition. Chemical and biological characteristics relate to conditions that may cause an accelerated or unusual corrosion problem with the cable or stabilization components and to marine organisms that constitute a hazard by attaching themselves to or boring into the cables. Seawater provides an environment in which corrosion of dissimilar metals in contact with each other can occur very rapidly. Some metals corrode at a relatively uniform rate that can be predicted quite accurately, while others, such as stainless steel, do not. The use of these nonpredictable materials should be avoided if possible. If not, a large factor of safety should be used when calculating the expected life of the immobilization system.

Sulfate-reducing bacteria in an anaerobic environment produce hydrogen sulfide which can accelerate the corrosion rate. Aerobic bacteria can cause organic material, such as tar in jute roving, to decompose (Cullison, 1975). These conditions are not often found in the nearshore region; however, they should be looked for in some bays and lagoons where water can stagnate.

The more common types of biological fouling found to affect cable installations are kelp and coral. Kelp, which is a species of algae that grows in large tufts, firmly attaches itself to rock or cable by means of numerous rhizoidal filaments called "holdfasts." Kelp has been observed at depths of 250 feet (80 m), but the heaviest growth seems to occur in less than 50 feet (15 m) (Sverdrup, 1946).

Coral reefs are a result of biological precipitation of calcium from seawater by corals. Reef-producing corals are found only in areas where water temperatures are above 20°C and are, therefore, confined to shallow water of tropical seas (Sverdrup, 1946). There are reports

of coral growth on cables located in tropical waters (Cullison, 1975), but usually the occurrences appear as small, isolated clumps, averaging about 6 inches (15 cm) in diameter.

The hydrodynamic effect of large amounts of kelp or coral attached to a cable is significant; however no known theory exists that allows accurate hydrodynamic modeling of fouled cables. In areas of very active coral growth, the cable may become completely encased in a coral formation, thus acquiring additional natural stabilization. Since the growth rate of coral is slow even in active areas (about 0.5 in./yr; 12 mm/yr), this mechanism can not be counted on as the only means of stabilizing the cable.

Data Requirements. Qualitative data on the type, amount, and distribution of marine growth in the area of the proposed cable route should be documented in the site survey report. Information must be obtained about the materials used in the cable to determine if any corrosion problems will occur with the materials used in the stabilization system.

Equipment Available to Gather Data. No specialized equipment is required.

Reporting. Data on the chemical and biological characteristics of the environment should be presented in narrative form in the site survey report.

2.2.6 Currents, Waves and Winds

Definitions. Currents are the movement of ocean water normally in a continuous stream flowing along a definable path. Waves are defined as "disturbances which move over the surface of the ocean with speeds dependent upon the properties of seawater" (Ciani, 1974). Winds are caused by moving air, especially a mass of air having a common direction of motion (Ciani, 1974).

Currents in the ocean are of three general types: wave generated, drift, and tidal. The speed of wave-generated currents is usually less than 1 knot (NAVFAC P-906, Vol I). Drift currents, which include inertial, geostrophic and wind-driven currents, vary up to 5 knots; this is the maximum current on the surface of the Gulf Stream (Myers et al., 1969).

Waves may be classified in terms of several major characteristics, including their period (or frequency), profile, horizontal motion (standing wave versus progressive), and height (NAVFAC P-906). In the nearshore region the most useful method of classification is by wave height and frequency. Assuming a sinusoidal profile, the shape of a wave, η, may be described by the equation:

$$\eta = \frac{H}{2}\cos\left(\frac{2\pi}{L}x + \frac{2\pi}{T}t\right) \qquad (2\text{-}1)$$

where H = wave height = 2A

T = period

L = wave length

d = water depth

t = time

x = distance from coordinate axis

When plotted, Equation 1 appears as the curve shown in Figure 2-4.

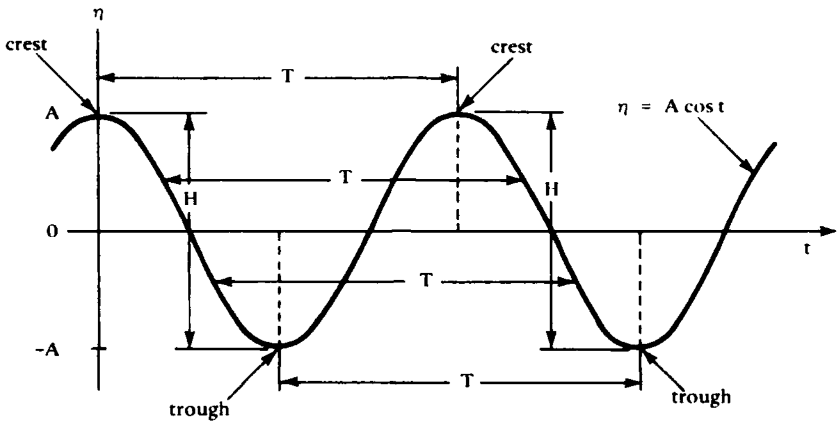


Figure 2-4. Sinusoidal shape of a wave.

As waves approach shallow water (water depth less than one-half the wave length), the height increases and the velocity decreases until the depth is approximately 1.3 times the wave height; at this point, the wave becomes unstable and breaks. The area between this point and the shore is the surfzone. If the water deepens again as the broken wave continues toward shore, it is possible for it to reform. This new wave will of course be smaller than the original wave, and will break in shallower water.

Winds are characterized by their speed and direction. Because wind speeds at any given time vary with height above ground or sea level, a convention was established to quote wind speeds at a height of 30 feet (10 m) whether over land or sea. Because the winds at any location vary with time, another convention was established for giving wind speeds. This convention is the "fastest mile of wind," which is the maximum speed of the wind averaged over 1 mile. The probable period of recurrence of winds of this maximum speed is also significant. The stated direction of the wind is that direction from which it blows, e.g., a west wind blows from the west to the east.

Maximum Design Conditions. The design of cable protection/immobilization systems requires an estimate of the loading condition that would be anticipated during the operational life of the in-place cable. For submerged cables this design condition would be defined in terms of the forces of currents and waves. Maximum design conditions for the installation of cable protection/immobilization systems are considered later in the section on Weather Window.

Maximum design conditions for current exist when its speed is greatest and its direction is perpendicular to the cable. The forces of currents are a function of the square of the speed. The maximum design conditions for ocean waves exist when the waves are highest and their periods shortest. As in the case of currents, the critical condition of wave direction is when this direction is perpendicular to the cable. The maximum design wave condition is a function of the site and historical data on waves at that site, particularly storm waves. For example, on the Gulf Coast of the United States, the deep water waves are normally less than 5 feet (1.5 m) high, but during the hurricane season (in the late summer and early fall) waves up to 80 feet (24 m) have been observed. Typically, the conditions of a 20-year storm (i.e., a storm that has the probability of recurring every 20 years) are used as maximum design data (NAVFAC P-906, Vol. I, p. 32). For the Gulf Coast example, the 20-year storm produces waves that are 66 feet (20 m) high.

Winds have little direct influence on submerged cables once they are installed (NAVFAC P-906, Vol. II, p. 29). Waves are most frequently the result of winds, but the mechanism of the wind generation of waves is beyond the scope of this discussion. Winds affect the installation of submerged cables, but these effects will be addressed later. The wind records required for the design of these systems are those for the fastest mile of wind for 30 feet above ground level with a return period greater than the life requirement of the system (Chapter 7). Further information on design wind conditions may be found in NAVFAC P-906 (Vol. II, pp. 35-39) and Myers et al. (1969).

Weather Window for Installation. Calm weather and sea conditions are desired for most cable system installations because unfavorable conditions may induce undesirable motions of the work platforms (NAVFAC P-906, Vol. I, p. 32) and excessive tensile loads on the cable. The installation of cable protection/immobilization systems, particularly split pipe, also requires mild conditions of currents, waves, and winds because these operations frequently involve small boats and divers operating in shallow water. The weather window may be defined as "the continuous time interval expected to be available during which the weather and sea state will not impede or halt operations. Weather window duration varies with the location, time of year, and the operation being performed" (Valent et al., 1975b). In identifying the time and duration of a weather window for any given operation at any location, the expected strength of the current, height and period of the waves, and speed of the winds are the primary considerations. The

directions of the currents, waves and winds are generally less important than the severity of these environmental conditions in identifying the weather window.

Currents are usually not the determining factor in identifying the weather window, but they are important considerations at a few sites. The velocity of the wind-driven currents are a function of the wind speed and fetch length.* The velocities of these currents vary seasonally, but seldom by more than a fraction of a knot and only then at sites with a long fetch. The velocities of the most rapid currents, tidal currents, may vary tremendously during the day in both speed and direction. The best time for operations in a tidal current is at slack tide when currents are theoretically zero. The strength of these currents, the times of maximum ebb and flood currents, and the times of the slacks at operational sites are predictable. Thus, the time of the weather window for currents is very site dependent in terms of the fetch length and tidal factors.

Waves, primarily those generated by wind, are a more significant factor than currents in determining the weather window. Wind waves are of two general types: sea waves, which are generated or sustained by the wind within the fetch length, and swell waves, those which have left the area in which they were originally generated. Waves of the sea type range in period from less than 1 second to over 15 seconds. The shorter period waves dissipate rapidly when they are not supported by the wind. The longer period waves persist as swell beyond the generating area because they can be sustained by gravity and do not need the force of the wind to maintain their integrity.

The height and period of wind-generated waves in shallow water (5 to 90 feet; 1.5 to 15 m deep) may be predicted using the curves given in the Army Coastal Engineering Research Center, Shore Protection Manual (Army CERC, 1973). Sea waves may also be defined in terms of a descriptive scale of sea states from 0 for a calm sea to 9 for storm conditions (Myers et al., 1969). The time and duration of waves of the sea type are primarily a function of the wind speed and direction at the time and place of the operation. As with the direct effects of the wind, which are described below, aperiodic storms must be carefully watched because they are not predictable but can have a tremendous effect on installation operations.

As discussed above, waves of the swell type exist in the absence of wind. The height and period of waves generated by wind in deep water and proceeding to shallow water as swell can be predicted using the procedures and curves given in Army CERC (1973). Thus, the time and duration of the weather window for swell can be predicted better than that for sea, but even this weather window cannot be identified much in advance of the operation.

*The horizontal distance (in the direction of the wind) over which the wind blows generating currents and waves.

Wind speed is the most significant consideration in defining the weather window for operations to install cable protection/immobilization systems. The indirect effects of the wind on these systems via their generation of currents and waves were discussed above. In addition to these indirect effects, the wind directly affects these operations by displacing the work platforms from which these systems are installed and divers are deployed. The weather window for the direct effects of the wind is defined by the predicted maximum wind speed (the fastest mile) and its direction (from which the wind is blowing). Prevailing winds at the installation site and the time of day that these are at a minimum is a factor in making advance predictions of the weather window. The usual dates of occurrence of seasonal storms are another factor. Aperiodic storms must be carefully watched shortly before and during operations because preliminary estimates of the weather window based on seasonal and time of day predictions may induce a false sense of security.

Data Requirements. The information required to anticipate either the maximum design conditions or the weather window must be taken from historical data. Measurements of currents, waves, and winds at any single time before an operation reveal little of interest to the planner of a cable protection/immobilization operation.

For maximum design conditions, data on maximum expected wave height, period and direction during the life of the cable must be determined. This is obtained by calculating the height of the maximum storm waves with a return period equal to or greater than the design life of the installation. Waves produced by 20-year storms have typically been used for design purposes. Data are also required on maximum current velocity and direction anticipated at the site. Data on wind speed are useful only in their effect on wave height. If wave height data are directly available, wind speed has no effect on the design of the stabilization system.

To determine the best time of the year and weather window for installation, data are required on wave height and direction, current velocity and direction, and wind speed as a function of the time of the year. If amphibious operations are anticipated, data are required on the size of the surfzone and number of rows of breaking waves that will be encountered.

The accuracy for wind, wave, and current data as established by Ciani et al. (1974) is presented in Table 2-2.

Table 2-2. Accuracy Requirements for Wind, Wave, and Current Data

Property	Accuracy
Wave Height	1 ft (0.3 m)
Wave Period	1 sec
Wave Direction	5 deg
Current Speed	1 ft (0.3 m)/sec
Current Direction	15 deg

Reporting. For maximum design conditions data on expected maximum wave height

and period are presented in tabular or graphical form as a function of direction. The data should also indicate the time of return of the storm used to derive the data. Current velocity data are also presented in tabular or graphical form as a function of direction.

Data for determining the optimum weather window should be presented in graph form, indicating percent of time during each month of the year that wave heights and wind velocities are expected to be below certain discrete levels. In areas where tidal currents could affect the installation operation, copies of tide charts or tables for the area should be obtained.

2.3 HAZARDS

A hazard is a natural or manmade condition or phenomenon that is potentially a source of damage to a cable system. Usually the time of occurrence and the extent of damage cannot be predicted for an individual case; however, overall probability can be assumed. General techniques for including hazardous conditions in the design of a cable system are: (1) to avoid the hazard, (2) to accept a calculated risk (e.g., a trade-off between operational criticality of the cable system versus the added cost of avoiding or protecting against the hazard), and (3) to provide protection for the assumed "worst case" during the cable system design life.

2.3.1 Anchors

Drag anchors can be a hazard to submarine cables because of the danger of their engaging cables, even buried ones, while being pulled horizontally during setting, retrieving, and dragging. Even heavily armored cables will probably be damaged as the anchor engages and slides along the cable; often the cable will part due to the combined cutting and tensile loading of the cable. Anchors may damage a cable if dropped directly on top of that cable; however, the probability of this "direct hit" occurrence is small compared to the probability of anchor/cable contact during setting, retrieval, and dragging of anchors.

Data Requirements. Input* required in determining the optimum depth of burial at a given site are:

(a) Anticipated frequency of a given vessel dragging anchor in terms of vessel size

(b) Anchor size corresponding to a given vessel

(c) Depth of anchor penetration corresponding to a given anchor type and size (see Table 2-3)

*Adapted from Brown (1971).

(d) Degree of cable damage as a function of anchor size and cable type (amount of armoring)

(e) Cost of cable repair as a function of cable type and depth of burial

(f) Cost of burial in terms of burial depth, cable type, and soil type

Table 2-3. Anchor Burial Depths (from Valent and Brackett, 1976)

Anchor Weight		Fluke Tip Burial Below Firm Bottom							
		Standard Stockless				Danforth or LWT			
		Sand		Mud		Sand		Mud	
lb	Mg	ft	m	ft	m	ft	m	ft	m
3,000	1.4	6	1.8	9	2.7	10	3.0	24	7.3
10,000	4.5	10	3.0	12	3.6	12	3.6	17	5.2
20,000	9.1	12	3.6	12	3.6	14	4.3	a	a
30,000	13.6	14	4.3	25	7.6	a	a	a	a

[a]No data.

The end result of this analysis is a comparison of total cost (initial capital cost plus anticipated maintenance and repair cost) versus depth of burial. From this comparison, a minimum cost is ascertained that corresponds to some burial depth at the given site. A comparison of minimum costs for the sites considered, or a comparison of risk to cable system mission, can then be used to select the best cable landing site.

Equipment Available to Gather Data. Information on seafloor material type and condition is necessary. It is obtainable through a cable route survey that includes some material sampling and a determination of soil cover thickness over rock. A vibrocorer or push or gravity corers can obtain the necessary soil samples, and a shallow water sub-bottom acoustic profiling device can define the soil/rock contact (as described by Ciani and Malloy, 1975). Additional equipment is discussed in Section 2.2.1.

The required accuracy of the data for determining anchor drag hazard is quite liberal. Only the soil material type need be identified with a coarse measurement of soil strength/density because anchor penetrations are tabulated by such general data (e.g., see Table 2-3). Soil cover thickness measurements (over rock) of ±3 feet (±1 m) are adequate for providing an answer as to whether the cable can be placed beneath the reach of a given anchor. In most situations, the data accuracy required for other necessary aspects of a given cable installation will exceed those accuracies required for drag anchor hazard analysis.

Reporting. Sediment cover over rock is commonly reported in graphical form as a profile of seafloor elevation and rock contact elevation referenced to a known sea level condition (see Figure 2-1). Soil

material types can be noted directly on these graphical profiles in the nomenclature of the Unified Soil Classification System (NAVFAC DM-7, 1971). Data on the rock material should include a visual classification of rock type (i.e., basalt, coral, beachrock, etc.), structural features of the rock mass, and rock strength. Such information will be necessary if trenching or rock bolting of a cable to an exposed rock seafloor is attempted.

2.3.2 Trawler Fouling

Fouling of ocean cables by trawl gear is not considered a nearshore cable problem, but it is mentioned here because trawlers may operate in waters as shallow as 120 feet (36 m) (NAVFAC, 1975). Damage can occur when cables are snagged by the heavy, metal-clad "otter boards" that are dragged along the seafloor to keep the mouth of the trawl net open (Figure 2-5). Scallop dredgers and clam dredgers also use equipment that penetrates the ocean bottom and could damage cables (Anon., 1967). Burial of cables to a depth of 2 feet (0.7 m) is apparently sufficient to eliminate much of the trawler damage (Anon., 1967).

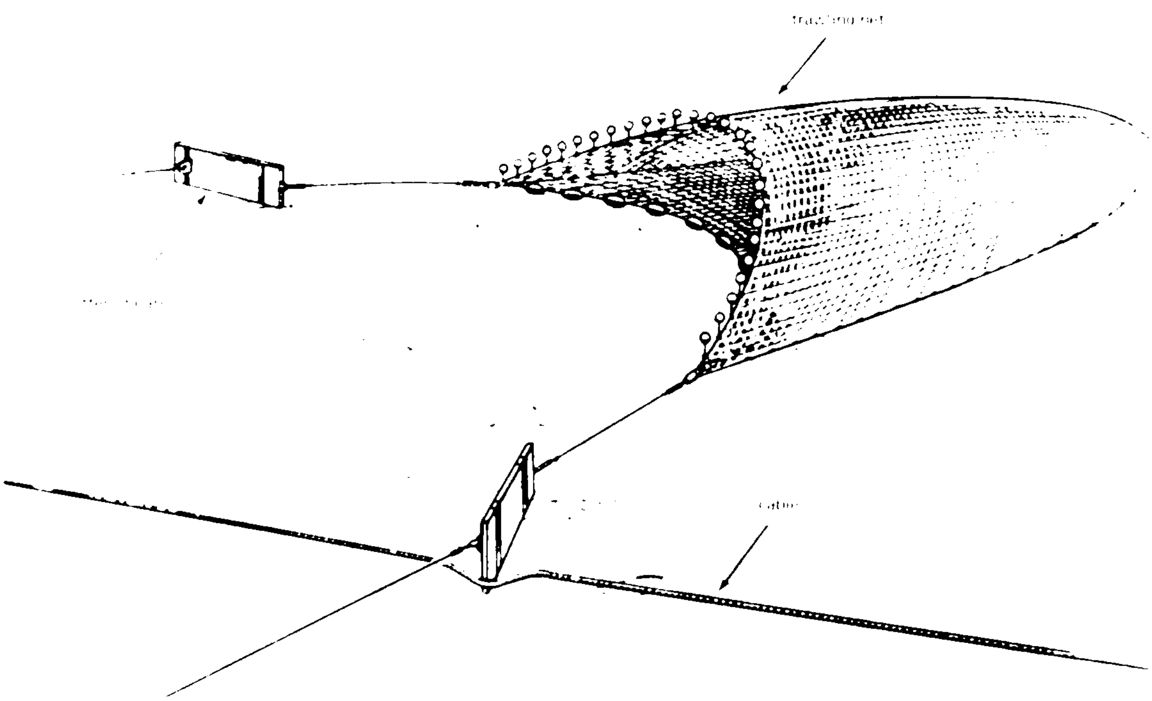


Figure 2-5. Bottom trawling configuration.

Data Requirements. Data are required to establish whether or not trawl fishing is practiced along the proposed cable route. Known seafloor conditions may assist in this determination; for example, areas of exposed seafloor rock will not be frequented by trawlers because of the likelihood of damaging their nets on the rocks. However, the local fishing industry is probably the best source of data on fishing intensity along a proposed cable route. The data need be accurate only with respect to location coordinates: ±0.5 mile (±1 km) in identifying fished versus non-fished areas.

Equipment Available to Gather Data. No specialized equipment is required. A routine cable route survey will indicate the presence of exposed rock.

Reporting. Areas of potential trawl fishing damage should be outlined on maps of the proposed cable routes to indicate those route increments requiring burial for protection.

2.3.3 Scouring

Scouring is the removal of the ocean floor soils by currents or wave action. Scouring can: (1) decrease the depth of burial of a cable, thereby reducing its protection against damage by dragging anchors or trawler otter boards; (2) expose a cable to abrasion by water-driven gravel or sand; (3) expose a cable to current and wave forces that cause it to grate on hard rock surfaces.

The occurrence of scour may be cyclic or continuous, and it may be natural or man-induced. An example of natural cyclic scour is the shifting of sands and gravels with the seasons that is common in nearshore areas. Problems have developed with cables that were buried during the summer in a shallow sand cover over rock; during the winter months the sand was removed, and the cable was left exposed to abrasion and to current and wave forces. In an extreme case, due to special circumstances, cyclical scour has removed 30 feet (9 m) of material (Griswold, 1975), but seasonal scour on the order of 6 feet (2 m) (Zenkovich, 1967) is probably more often the case. An example of a natural continuous process is that of a retrograding shoreline or that of the changing course of a river, especially at flood stage. In either instance, the cover over a cable could be reduced or eliminated, exposing that cable to damaging forces and abrasion. An example of man-induced scour is that of an excavated trench in hard materials where the current and wave forces were sufficient to remove the unconsolidated backfill in the trench.

Data Requirements. The input required is data on the maximum depth of material expected to be removed by scour along a given cable route. Then, if the cable is to be kept buried during its design life, it must be buried below this profile of maximum scour depth. Data can be obtained by surveying a given cable route twice during a 1-year

duration, once shortly after a period of major storms and once toward the end of a major cyclic period of calm weather. This will yield hopefully the extremes of sediment levels.

It is particularly important to identify the existence of subsurface hard, consolidated soil strata, gravel, cobble, boulder layers, or bedrock that could potentially be exposed by scour. Ocean cables will generally not suffer problems when subjected to scour on a sand seafloor because the heavy, armored shore cable will sink some short distance below the sand surface under the influence of wave action if sufficient slack is left in the cable to allow it to sink as the sand moves from beneath it. If, however, all of the sand cover over a hard seafloor strata is removed, then the cable will be directly subject to wave and current forces and to abrasion by wave-driven gravel, etc.

If the initial acoustic and visual survey indicates a cover of sand over hard seafloor strata of such thickness that scour would not normally be expected to remove the total sand interval, then further surveys of seafloor profile variation with season are generally unnecessary. The determination of maximum potential scour depth should be obtained from an experienced coastal engineer. The expected variation in scour depth from year to year can be considerable, thus the accuracy of measurement in any one year need be only to the nearest 3 feet (1 m).

Equipment Available to Gather Data. A shallow water sub-bottom acoustic profile can define the seafloor profile and the soil/rock contact profile (as described by Ciani and Malloy, 1975).

Reporting. Data should be reported as a profile of seafloor elevation and rock contact elevation referenced to a known sea level condition. Predicted or projected scour elevations should appear in the same format (Figure 2-6).

2.3.4 Ice

Ice is a hazard to submarine cables from the beach to water depths of 650 feet (200 m) because of the tremendous forces applied: (1) by floating ice cutting into and scoring the shallow ocean floor and shoreline or grounding on exposed seafloor rock, and (2) by fast ice with cables included in the frozen mass breaking loose from the seafloor or shore.

Observation of score depth versus water depth for two locations with soil seafloors are given in Table 2-4 (Hironaka, 1974). On rocky seacoasts, notably the south coast of Greenland, cable damage due to flattening or crushing by grounding icebergs is reported to 625 feet (190 m) (Myers et al., 1969).

Armoring or strengthening of electrical cables to resist damage by grounding icebergs, rafted ice, and pressure ridges is impractical at best. Protection of cables against ice damage is best considered a balance between the cost of avoiding potential damage (by deeply burying the cable or by installing the cable in a drillhole) and the cost of cable repair or replacement in more vulnerable designs.

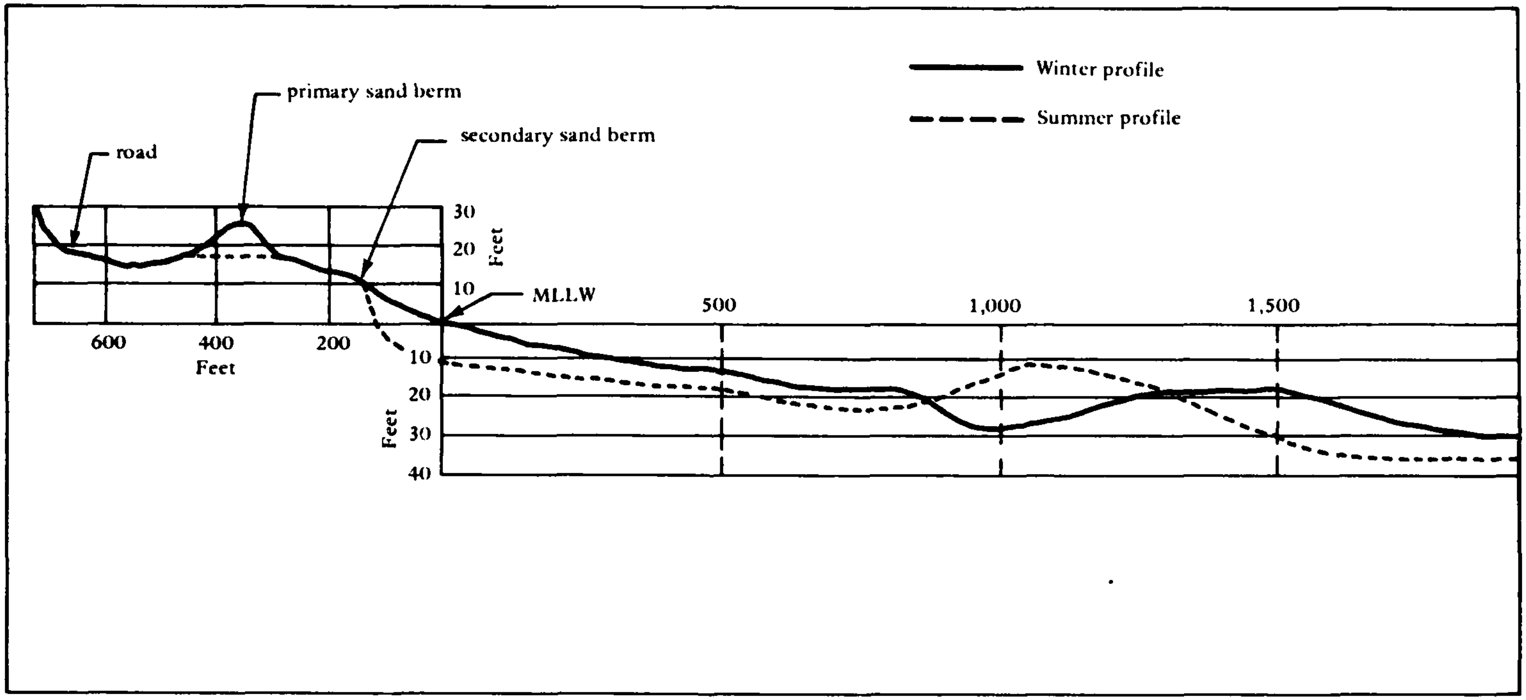


Figure 2-6. Inshore and beach profile for summer and winter.

Table 2-4. Ice Scoring Penetrations

Location	Maximum Score Depth		Water Depth Range		Frequency
	ft	m	ft	m	
Beaufort Sea Shelf	2	0.6	0-20	0-6	high
Beaufort Sea Shelf	15[a]	4.6[a]	20-100	6-30	moderate
Beaufort Sea Shelf	30	9.1	100-250	30-80	low
Between Harrison Bay and Barter Island, Alaska	7 (2-3 avg)	2.1 (0.6-1 avg)	50-200	15-60	–

[a] Usually less than 5 ft (1.5 m).

Data Requirements. The most significant data are those describing the degree of extent of ice action on the seafloor. Data are required on: (a) the average and maximum probable depths of ice keel penetration (score depths) as a function of water depth along the proposed cable track, and (b) the time frequency of occurrence of average and maximum depths of scoring as a function of water depth. Side scan sonar equipment has been used to obtain score depths versus water depths (Kovacs and Mellor, 1974), but the time frequency of occurrence is not so readily available. Based on available inconclusive evidence, it appears that the deepest scores can be expected to occur annually at a given cable route (interpreted from Kovacs et al., 1973, and Kovacs, 1972). Scoring probably no longer occurs in the Beaufort Sea at water depths over 45 meters (Kovacs, 1972); scores at deeper depths are thought to be relict.

These data on probable score depth specifies the cable profile required to avoid damage from scoring ice. To evaluate the technical feasibility of burying a cable below a specified depth, information on the seafloor material down to that depth is necessary. If a horizontal drillhole is being considered as a means to avoid scoring problems, then of course material properties from those deeper strata through which the drillhole will pass are required. For cable burial, classification of a soil is required - cohesive versus noncohesive, grain size distribution, strength index, etc. - in order to identify the best burial alternative, i.e., plowing, jetting, trenching, etc. Classification of the soil is also necessary to determine the feasibility of burying the cable to the depth required. If the scoring ice is predicted to reach hard material, i.e., rock, both material (i.e., shear strength, etc.) and mass properties (jointing, bedding, etc.) are required. In arctic regions, data on the permafrost (meaning "frozen soil" rather than material with a temperature less than 0°C) distribution will be required to permit proper

selection of trenching equipment or design of drillhole casing thermal protection. Depth of scores should be measured to the nearest 1 foot (0.3 m). The former depth of a soil or rock specimen in the seafloor profile should be identified to the nearest 1 foot (0.3 m).

Equipment Available to Gather Data. Bottom profile information for the mapping of scores can be obtained with available side scan sonar equipment during periods of open water, along with sub-bottom profile data from sub-bottom sounding equipment to determine the proximity of hard layers. Available gravity corers can obtain short sediment cores of sufficient quality. Samples of deeper material for evaluation of deep trench or horizontal drillhole feasibility may have to be obtained from the surface of the shorefast ice via conventional terrestrial arctic soil boring and sampling techniques.

When cable burial via plowing, jetting, or trenching is selected, a preliminary pass along the proposed cable track should be made to ensure that no unforeseen impediments to machine operation and cable burial to required depth exist. (Note: Available weather windows may be too short to permit the luxury of such verification.)

Reporting. Expected depth of ice mass scoring can be indicated on the profile of seafloor topography, sediment strata, soil/rock contact topography and rock strata.

2.3.5 Marine Organisms

Marine organisms generally are not a problem with present polyethylene-insulated cables. Teredo attack on these cables can be invited, however, by allowing materials susceptible to teredo attack to remain attached to the installed cable (materials such as manila line, untarred hemp-spun yarn, and canvas parceling (Myers et al., 1969). There is one reported instance of a mollusc attaching itself to a communications cable and by some undefined mechanism penetrating through to both conductors (Snow, 1974).

Since the net effect of marine organisms on the performance of a well-designed cable system is minimal, no further discussion of this potential hazard will be made.

2.3.6 Earth-Mass Movements

Earth-mass movements may be of several forms and have a number of different causes. Earth-mass movements range from the simple sliding of one face of a fault zone relative to the other (usually only a few meters relative displacement), to the slumping or sliding of blocks or sheets of soil or rock (displacing a few hundred meters), to the flow slide and turbidity current (where material may move one to a few hundred kilometers). Earth masses are often brought to a condition of near incipient motion by natural processes, and then are triggered into motion by an external event. Natural processes acting to decrease the

stability of earth masses include oversteepening by scour and wave action and deposition of material at the top of river deltas or the heads of submarine canyons (Terzaghi, 1956). Triggering mechanisms include earthquake vibrations, wave forces (Henkel, 1970; Bea and Arnold, 1973), manmade shock waves and man's construction activity on the seafloor.

Cable failure occurs because the cables are caught up and included in the moving earth mass, thereby creating excessive tension in the cable. Cables that happen to traverse the initially unstable earth mass, even if buried only 3 feet (1 m) deep, would undoubtedly be broken by any sizeable earth movement. Cables crossing the path of such a moving earth mass could conceivably be protected by shallow burial; however, the nature of submarine earth-mass movements, especially flow slides and turbidity currents, and the extent and depth of disturbance of the underlying seafloor are not sufficiently understood to support recommendations. Problem solution in the nearshore appears best achieved by good site survey, identification of potentially unstable earth-mass areas, and cable re-routing to avoid those areas. A generalization by the authors based on the observations of others is that the following areas should be treated as potentially unsafe when selecting nearshore cable landing routes:

(a) In nonseismic areas, underwater slopes of over 0.07 radian (4 degrees) (Morgenstern, 1967)

(b) In seismic areas, underwater slopes of over 0.04 radian (2.5 degrees) (Jacobi, 1976)

(c) In rapidly accumulating delta areas, underwater slopes of over 0.01 radian (0.5 degrees) (Henkel, 1970; Bea and Arnold, 1973)

Data Requirements. Areas of potential earth-mass movement must be identified and properly delineated. Possibly the best source of data will be a recent historical record of nearby areas; i.e., is there any record of underwater landslides or even coastal landslides that have entered the water? If the potential for earth-mass movements is suspected by (a) history, (b) high risk of earthquake occurrence, (c) high sedimentation rates, and/or (d) steep underwater slopes, then a detailed cable route survey by experienced personnel should be authorized to delineate the extent and magnitude of the problem.

Bottom and sub-bottom topography and sediment shear strength and sensitivity data should be obtained (see Section 2.2.1). Bottom and sub-bottom elevations for slope stability assessment should be measured to the nearest 1 foot (0.3 m).

Equipment Available to Gather Data. A gravity corer or vibrocorer can obtain the necessary soil samples, and a shallow water sub-bottom acoustic profiling device can define strata.

Reporting. There is no set way to report data on the potential for earth-mass movement because the data are not amenable to a set format. What is required, if earth-mass movements are found to be a potential problem along a given route, is a map of probable movement-affected areas, including the head, probable travel path, and probable terminus. Cable routes having the least degree of exposure to these probable earth movements can then be laid out.

2.4 OPERATIONAL SUPPORT

Operational support parameters include quantitative and qualitative data that describe all of the vital functions required for successful completion of the operation. Most of these factors can be classified under two major categories: (1) facilities and (2) logistics.

2.4.1 Facilities

Facilities consist of all structures, spaces, and fixed equipment required to support personnel and equipment during the installation of the stabilization system. These facilities include as a minimum: (1) structures for messing and berthing of personnel, (2) an area for storage of equipment and materials, (3) an area for maintenance and repair of equipment and vehicles, (4) harbor facilities, and (5) facilities for washing, drying, and storing diving equipment (if required). Some stabilization techniques require additional specialized facilities. These will be included in the discussion of the individual stabilization technique.

Data Requirements. Information on the size, location with respect to the work site, number of personnel that can be accommodated (i.e., for messing and berthing), and significant features should be obtained for each of the facilities that may be required to support the operation. The capabilities of specialized facilities, such as machine shops, auto repair facilities, etc., should be documented. If harbor facilities are required, data on docking facilities and maximum draft of ships that can be taken into the harbor must be obtained.

Reporting. There are several acceptable methods of reporting on facilities. Data may either be reported in narrative form or pertinent data may be noted on a chart of the area that shows the location of various facilities (Figure 2-7). A photographic record of specialized facilities is often useful as a supplement to these types of reports.

Pages 2-27 and 2-28
not available

Method of Gathering Data. Data on logistics support requirements will be generated from the design of the cable stabilization system and from observations and discussions during the on-site investigation. The exact sources of logistics support information will vary from site to site, but generally data can be obtained from maps and charts of the area, discussions with the Public Works Officer (if there is a military installation located near or at the site), and contact with local supply officers to determine methods and cost of shipping equipment and materials.

Reporting. Logistics support requirements will normally be reported in a project execution plan generated after the stabilization system has been selected and designed.

2.5 MISSION-RELATED PARAMETERS

Mission-related parameters are those characteristics that are not dependent on any particular site location, but define the operational requirements of the system. Information concerning these parameters will normally be furnished by the mission sponsor in a statement of work or project order. In a few instances the design of the stabilization system may require alteration of these parameters. The parameters identified as being mission-related include: (1) system design life, (2) criticalness of the system, (3) type of cables, (4) number of cables, (5) length of protected cables, (6) corridor width, and (7) depth of burial. Two of these parameters, length of protection and depth of burial, will generally be defined during the stabilization design rather than specified in the mission requirement statement.

2.5.1 Design Life of Cable System

The design life is the length of time during which the system is required to perform its planned function in the ocean environment. The design life of cable systems are generally classed in three ranges: (1) 1 to 5 years for research installations, (2) 5 to 10 years for oceanographic instrumentation systems and some test ranges, and (3) greater than 20 years for power, communication, and strategic defense installations. However, these ranges are not absolute; each installation will have its own design life requirement specified by the user.

2.5.2 Criticalness of System

The criticalness of the system is determined by the relative importance attached to the cable system continuing to provide the required power/data link for the specified lifetime of the system without interruption. Although not strictly related, it has been found that highly critical systems most often have the longest lifetime requirement.

Qualitative information about the criticalness of the installation will be supplied or can be obtained from the sponsor or system user. It is then up to the design engineer to use this information to establish the

factor of safety to be used in the design calculations. For those installations where lifetime and criticalness of the system coincide, it might be appropriate to establish the safety factor as follows:

$$S.F. = \frac{\text{Required Lifetime}}{2}$$

For installations in which these two parameters are not related, the stabilization design engineer will have to establish the safety factor based on judgment, past experience, and economics.

2.5.3 Type of Cable

Two types of cables are typically found in the nearshore region: (1) communications and signal cables, and (2) power cables. In some instances, the cable will be constructed to provide both functions. A typical cable consists of (1) the central core of conductors, (2) insulating material, (3) filter material (between twisted conductors), (4) electrical or magnetic shielding (optional), (5) steel armor for both abrasion protection and strength, and (6) an outside protective cover made of tarred-jute, rubber or thermoplastic. Figure 2-8 shows the construction of a typical nearshore coaxial cable.

The biggest variation in cables is in the central core of conductors. Communications and signal cables are generally coaxial or quad construction. Quads are produced by taking four individually insulated conductors and twisting them into a bundle. Construction of cables containing as many as 48 quads (192 conductors) are within the capabilities of most cable manufacturing companies. When the transmitted signal is in the high frequency range, coaxial cables are usually selected instead of quad cables. Power cables usually consist of 1 to 4 individually insulated copper conductors that may have a cross section as large as 1.6 inches (42 mm) (Myers et al., 1969). The type of cable selected for an installation will be affected by the design life and criticalness of the system as well as the power and signal transmission requirements of the facilities at both ends of the cable.

Physical characteristics of the cable to be used can be obtained from the cable manufacturer or system user. The characteristics most important to the design and installation of a stabilization system are:

(a) Weight - in air and underwater

(b) Minimum bend radius

(c) Breaking strength

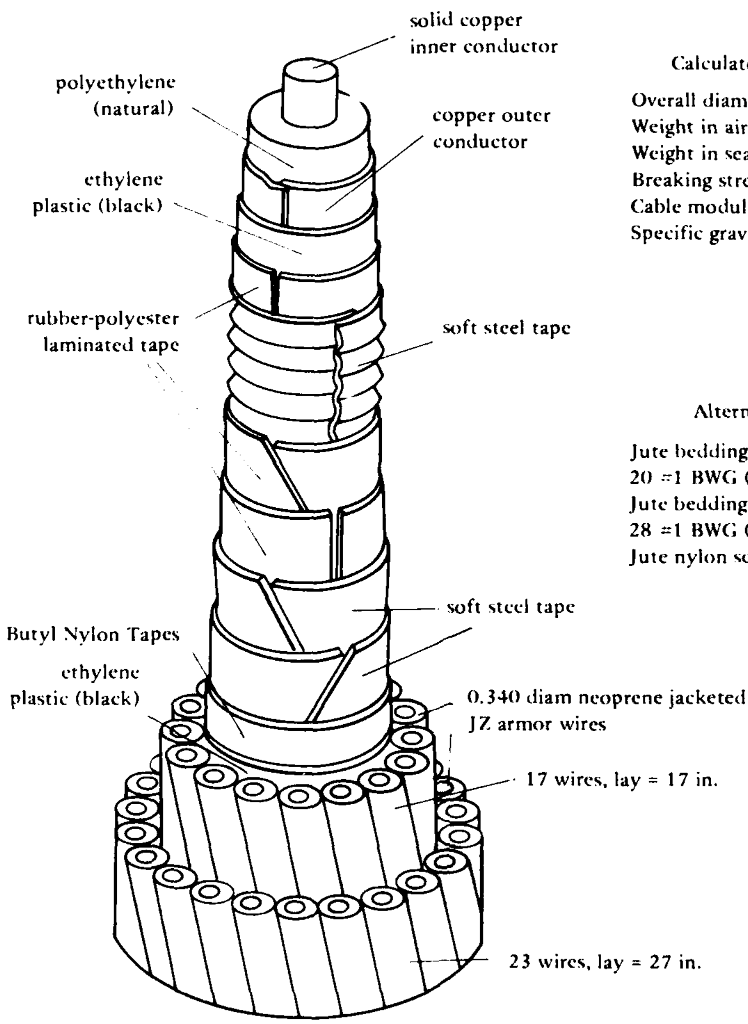
(d) Safe working strength

(e) Diameter

(f) Type and number of armor layers

(g) Type of outer protective cover

If the stabilization system is to be added to a previously installed cable, the condition of the cable must be determined, and conditions that might adversely affect the stabilization operation noted. Items to be investigated include:



Calculated Data for List 5 Cable

Overall diameter	3.50 in.
Weight in air	89,500 lb/naut. mi.
Weight in seawater	69,500 lb/naut. mi.
Breaking strength	168,900 lb
Cable modulus	2.43 naut. mi.
Specific gravity	4.48

Alternate Armor Detail

Jute bedding
20 #1 BWG (0.300" diam) armor wires
Jute bedding
28 #1 BWG (0.300" diam) armor wires
Jute nylon serves

Figure 2-8. Nearshore coaxial cable.

(a) Worn or damaged areas of the outer protective coating or armor wires

(b) Bird-caged armor

(c) Sections of cable that are buried

(d) Sharp bend, suspensions, or excessive slack in the cable

(e) Existing configuration, including splices and stabilization hardware

2.5.4 Number of Cables

The number of cables refers to the quantity of discrete cables approaching a common shore terminus from one or more seafloor locations. The number of cables along with the minimum spacing between the cables will determine the corridor width requirements.

2.5.5 Length of Protected Cable

The length of protected cable is defined as the total length of cable that must be stabilized and/or immobilized to prevent damage by wave or current-induced motion, or to avoid mechanical damage due to submarine hazards. The length of cable to be protected is also affected by the type of hazards that are found in the area (Section 2.3) and the logistics support available to get the required equipment to the site.

The required length of protection is determined in two steps. The first, a "Ball Park Estimate," is based on topography (Section 2.2.2) and expected maximum wave height (Section 2.2.6). The second, a detailed design, is based on the results of the hydrodynamic analysis discussed in Chapters 5 and 6. During the site survey, a ball park estimate of the protected length is all that is normally required. This estimate must be made prior to the completion of the survey to assure the survey covers the entire area where cable protection must be supplied.

Data Requirements. For the initial estimate on protection length requirements, data on topography and expected maximum wave heights must be obtained as outlined in the respective sections discussing these parameters. The maximum water depth to which protection must be extended may then be estimated using Equations 7-4 and 7-5 in Section 7.2.5. With the maximum water depth identified, the length of protection can be estimated by locating this depth along the proposed cable route on the topographic chart discussed in Section 2.2.2. The accuracy of specifying the length of protected cable will depend on the accuracy of the topographic and wave height data. Utilizing the procedures discussed in Sections 2.2.2 and 2.2.6, the protected cable length should be able to be estimated to within 30 feet (10 m).

Equipment Available to Gather Data. The equipment available to determine the topographic profile and significant wave heights is discussed in Sections 2.2.2 and 2.2.6, respectively.

Reporting. No formal reporting procedure is specified for this parameter. The length of cable requiring protection should be entered as a cable stabilization design requirement.

2.5.6 Corridor Width

Corridor width is the minimum horizontal distance perpendicular to the cable path required to accommodate the cable system. The corridor width requirements are a function of the type of cable, the amount of shielding, and the number of cables in the system (see Sections 2.5.3 and 2.5.4). Data on minimum cable spacing and number of cables are required to calculate the corridor width requirements. These data are obtained from the cable manufacturer and system user. Corridor width calculations can easily be specified to an accuracy within ±1.5 feet (±0.5 m); however, the ability to install a cable to this accuracy along the entire route is doubtful. A more realistic figure, even under ideal conditions, would be about ±30 feet (±10 m), which gives a minimum corridor width of 60 feet (20 m). This width may have to be expanded as the number of cables increases.

2.5.7 Depth of Burial

The depth of burial is the vertical distance of the cable below the seafloor. The distance is measured from the seafloor surface to the top of the cable. Burial is required for cables that must be installed in areas where hazards might penetrate the surface of the seafloor (see Section 2.3). The depth of burial is related to the types and frequency of hazards anticipated for a specific site (Section 2.3) and the type of bottom material found at that site (Section 2.2.1). The data requirements are discussed in Section 2.3, Hazards.

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Chapter 3

SITE ANALYSIS AND PRELIMINARY SELECTION OF FEASIBLE CABLE PROTECTION TECHNIQUES

3.1 INTRODUCTION

The previous chapter presented a discussion on the parameters that affect the selection, design, and installation of a cable protection system. In this chapter the manner in which these parameters influence the selection of individual cable protection techniques is discussed, and a series of decision matrices are presented to assist in eliminating the technically unfeasible protection techniques from the design process. In addition to obvious physical limitations imposed by the environment (e.g., inability to utilize expansion rockbolts in sand seafloors), the results (successes, failures) of previous cable protection operations have been incorporated into the decision matrix evaluation in an attempt to weed out those techniques that may be technically feasible but difficult to implement or economically unattractive because of conditions at the specific site.

The intent of this chapter is (1) to provide a basic understanding of the way in which each of the parameters influences the various systems and, (2) to provide a preliminary screening process that reduces to a reasonable level the number of protection techniques which are carried through the design phase.

Since it is impractical to consider all of the possible combinations of conditions that might exist at a particular site of interest, the decision matrices should be considered a design aid, applicable to a majority of the environments that are likely to be encountered. In the event that unusual site characteristics are encountered which make this screening process unsuitable (i.e., elimination of all potential techniques), then the decision as to which techniques are most feasible for the particular situation must be determined by reviewing the information presented for each technique in Chapter 4.

3.2 EFFECT OF VARIOUS PARAMETERS ON DIFFERENT CABLE PROTECTION TECHNIQUES

3.2.1 Bottom Material

The type of seafloor along the cable route will determine what types of stabilization techniques and what types of equipment will be functional in that particular environment. For example, on deep, clean,

sand seafloors, cables are often sufficiently stabilized by heavily weighting them with split pipe so that they sink, due to wave action, below the sediment surface. In cohesive soils (e.g., clays, muds), such sinkage might not occur, thereby requiring a trench to be excavated and the cable inserted. If deep burial in noncohesive soils is necessary, say to place the cable below the grade of future channel dredging, then specialized jetting stingers can be used to erode and suspend the sand in a slot and to insert a cable in that slot.

When the route passes over exposed or shallowly buried rock, the cable will often require protection from abrasion and damage by immobilizing it on the rock surface, or by placing it in a cut or blasted trench or in a drill-hole out of reach of waves and currents. The type and mass nature of the rock along the cable route will determine which protection/immobilization techniques and what type of equipment will best function in that particular environment.

If the rock is hard and massive, as for example a basaltic area, then existing mechanical trenchers will usually prove too slow to be an economic protection technique; either the cable will have to be weighted and bolted to the rock surface and/or a trench will have to be blasted for the cable. Alternately, if the horizontal traverse is sufficiently short and if ice damage is a continuing problem, then installation of the cable(s) in a horizontal drill-hole passing beneath the affected zone should be considered (Valent and Brackett, 1976). The occurrence of thin pockets of sand on rock or areas of broken rock and boulders will often necessitate re-routing of the cable in order to avoid potential problems through trying to reach sound rock for installation. In softer rock, mechanical trenchers will often prove better than blasting, especially in terms of environmental impact. The use of a drill-hole, however, becomes nearly infeasible because of the very long horizontal traverses usually associated with such materials and because of the increasing probability of encountering sticking or caving materials in the drill-hole.

Rock characteristics and properties determine not only what technique will be used to prepare a safe resting place for the cable, but also determine what hardware will be used to immobilize the cable. Conventional rockbolts work fine in hard rock applications; however, in coral, which is softer and much more porous than basalt, special rock bolts with very large expanding heads have proven necessary (Brackett and Parisi, 1975). Should the cable route traverse areas of boulders or small blocky rock, then anchoring to such small rock masses would probably prove quite ineffective for immobilization.

3.2.2 Topography

Seafloor and beach topographies have a significant impact on both the selection and implementation of the various cable protection techniques. This parameter also affects the selection of the most desirable cable route and the hydrodynamic analysis of the system because of its influence on design wave parameters due to shoaling and refraction.

Irregular seafloor topography will adversely influence the selection of techniques requiring large bottom-crawling equipment, such as mechanical trenchers, tracked drills, and some jetting systems. Sites where large suspensions or shifting topography (scouring) cannot be avoided may also rule out the use of rigid (oil field) pipe or pretensioning of the cable.

Beach topography will have little effect on the feasibility of any of the techniques, but it may influence the equipment used for installation or require extensive preparation prior to commencing the operation. Short, steep beaches will adversely affect the implementation of oilfield pipe and shore-applied split pipe installation operations. Beaches that do not have a convenient access route from the shore or where the shoreline topography is very rugged and irregular will influence the economics of the installation by eliminating the possibility of supporting the operation from shore.

3.2.3 Underwater Visibility

The visibility of underwater objects from the surface has very little impact on the selection of techniques for stabilization. Good visibility from the surface is always beneficial during the deployment phase of any of the mass anchor stabilization techniques because it allows placement of the chain, split pipe, etc. close to the cable, thus reducing the work required of the diver. The effects of surface water roughness and sun reflection can be eliminated by using a plexiglass viewing box.

Underwater visibility may have an economic impact on the selection of stabilization techniques. Visibility less than 15 feet (5 m) will generally rule out the use of large diver-operated machinery, such as tracked drills and seafloor trenchers, especially when combined with a rugged or irregular topography. Visibility less than 1 foot (0.3 m) will severely limit the efficiency of divers and reduce the cost effectiveness of any technique requiring diver installation.

3.2.4 Water Depth

Water depth will have the greatest impact on stabilization techniques requiring diver support. Figure 3-1 shows the no decompression time limits for divers based on the Navy diving tables. For depths greater than 60 feet (18.5 m) the amount of useful bottom time per diver during a normal work day is very short (less than 60 minutes). Therefore, installations that involve a lot of diving at depths greater than 60 feet (18.5 m) will require deployment of a larger number of divers.

The water depth profile will also determine how close to shore surface support craft can come and the feasibility of conducting amphibious operations from the beach.

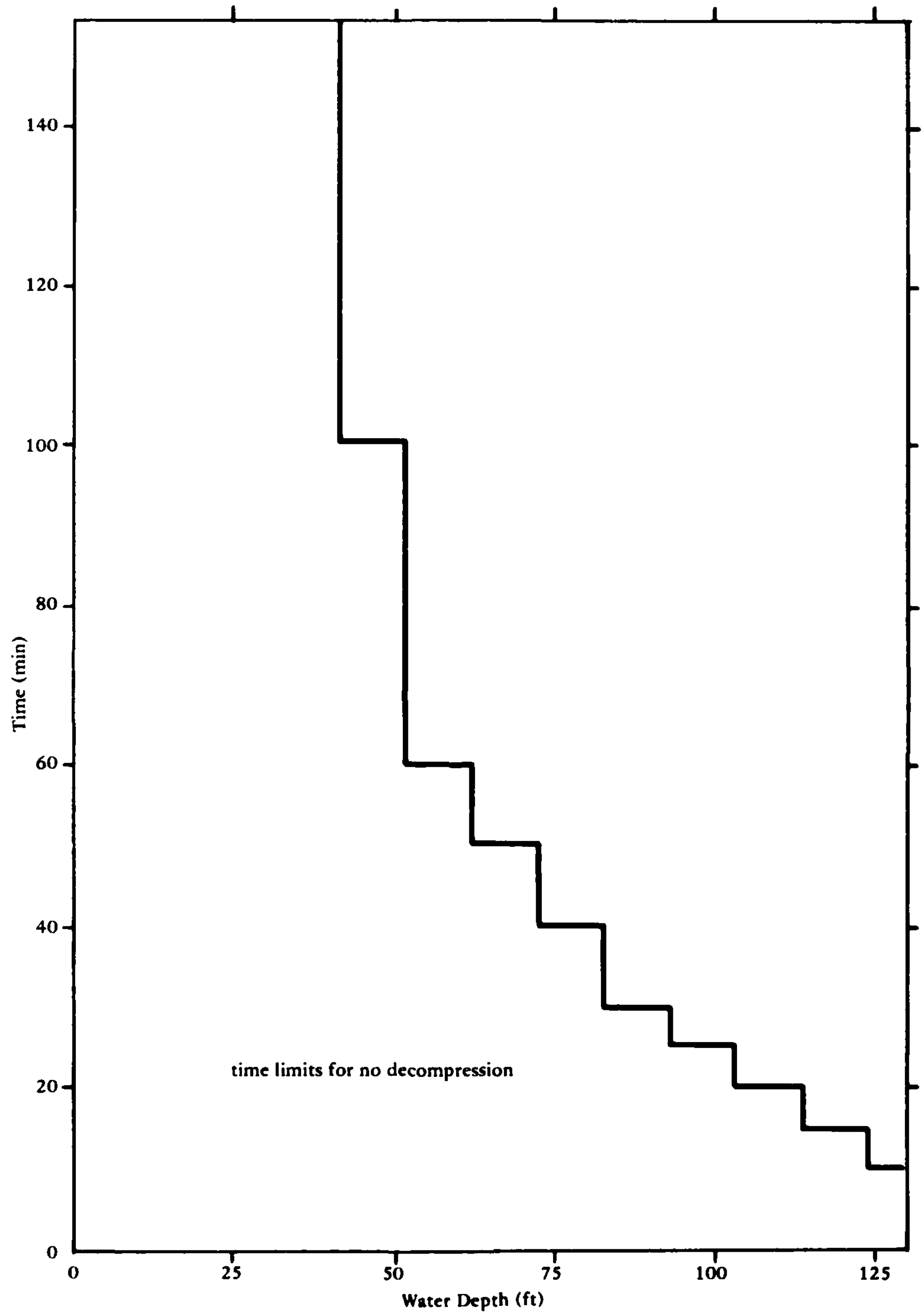

Figure 3-1. Diving limits for no decompression.

3.2.5 Chemical and Biological Characteristics

The presence of potentially harmful chemical and/or biological elements in the environment will affect the cable stabilization design rather than impacting on the selection or elimination of a specific technique.

Seawater provides an environment in which corrosion of dissimilar metals in contact with each other can occur very rapidly. Some metals corrode at a relatively uniform rate that can be predicted quite accurately, while others, such as stainless steel, do not. The use of these nonpredictable materials or dissimilar metals in contact with each other should be avoided, if possible. If not, a large factor of safety must be used when calculating the expected life of the immobilization system.

Sulfate-reducing bacteria in an anaerobic environment produce hydrogen sulfide which can accelerate the corrosion rate. Aerobic bacteria can cause organic material, such as tar in jute roving, to decompose (Cullison, 1975). Although these conditions are not often found in the nearshore region they should be looked for in some bays and lagoons where water can stagnate.

The more common types of biological fouling found to affect cable installations are kelp and coral. Kelp, which is a species of algae that grows in large tufts, firmly attaches itself to rock or cable by means of numerous rhizoidla filaments called "holdfasts." Kelp has been observed at depths of 250 feet (80 m), but the heaviest growth seems to occur in less than 50 feet (15 m) (Sverdrup, 1946).

Coral reefs are a result of biological precipitation of calcium from seawater by corals. Reef-producing corals are found only in areas where water temperatures are above 68°F (20°C) and are, therefore, confined to shallow water of tropical seas (Sverdrup, 1946). There are reports of coral growth on cables located in tropical waters (Cullison, 1975), but usually the occurrences appear as small, isolated clumps, averaging about 6 inches (15 cm) in diameter.

The hydrodynamic effect of large amounts of kelp or coral attached to a cable is significant; however, no known theory exists that allows accurate hydrodynamic modeling of fouled cables. In areas of very active coral growth, the cable may become completely encased in a coral formation, thus acquiring additional natural stabilization. Since the growth rate of coral is slow even in active areas (about 0.5 in./yr; 12 mm/yr), this mechanism cannot be counted on as the only means of stabilizing the cable.

In areas where heavy fouling is anticipated during the life of the installation, the added hydrodynamic force created by the marine growth must be accounted for in the design to insure the integrity of the stabilization/immobilization system.

3.2.6 Wind, Waves, and Currents

The maximum conditions of wind, waves, and currents expected during the life of the system are the basis for designing the stabilization/immobilization systems. The effect of these parameters on

the design and methods of predicting the maximum design condition are discussed in Chapters 5 and 7. These maximum values have very little influence, however, on the initial selection of technically feasible systems. The average annual minimum conditions, however, are extremely important in determining the feasibility of installing the various cable protection systems at a particular site.

Calm weather and sea conditions are desired for most cable system installations because high winds or rough seas can induce undesirable motions of the work platforms and excessive tensile loads on the cable. The installation of cable protection/immobilization systems, particularly mass anchor systems, also requires mild conditions of currents, waves, and winds, because these operations frequently involve small boats, deployment and handling of heavy objects, and divers operating in shallow water. The duration of these minimum conditions, commonly referred to as the weather window, is defined as the "continuous time interval expected to be available during which the weather and sea state will not impede or halt operations." Weather window duration and intensity of the minimum conditions vary with the location, time of year, and the operation being performed.

Currents are usually not the determining factor in identifying the weather window, but they are important at sites that have tidal currents that vary tremendously during the day in both speed and direction.

When currents in the work area exceed 1 knot, diving operations become difficult and excessive catenaries can develop in cables that must be floated on the surface for any length of time during installation. Current velocities that exceed 1-1/2 to 2 knots can be hazardous for nontethered divers, and production rates for operations requiring divers decrease to a level that usually make them uneconomical.

Waves, primarily those generated by wind, are a more significant factor than currents in determining the weather window. Wind waves are of two general types: sea waves, which are generated or sustained by the wind within the fetch length, and swell, which are waves that have left the area in which they were originally generated. Waves affect installation operations in two ways. The surge produced by passing waves influences the feasibility of diving operations in a manner similar to currents by making it difficult for the diver to move about and work on the seafloor. Large swell can produce undesirable heave and pitch motions of surface support ships, thereby making it difficult and sometimes dangerous to deploy heavy objects (mass anchor systems) or to conduct diving operations. However, absolute values for maximum swell height are difficult to establish because their effect on the operation will also depend on the waves' length and size and type of ship.

Wind speed is the most significant factor when defining the weather window for operations to install cable protection/immobilization systems. Wind has an indirect effect on these systems via the generation of currents and waves. In addition, it will directly affect these operations by displacing the work platforms from which these systems are handled and divers are deployed. The weather window for the direct effects of

the wind is defined by the predicted maximum wind speed (the fastest mile) and the direction (from which the wind is blowing). The prevailing winds at the installation site and the time of day that these are at a minimum are factors in making advance predictions of the weather window. The usual date of occurrence of seasonal storms is another factor. Aperiodic storms must be carefully watched shortly before and during operations, because preliminary estimates of the weather window time and duration based on the above seasonal and time of day predictions may induce a false sense of security.

3.2.7 Anchors

In soil seafloors, cables cannot economically be buried sufficiently deep to place them below the limit of all possible anchor penetration. Rather, there is an optimum depth of burial in terms of risk at a given site; alternatively, another site with a lesser risk of anchor drag damage may be considered.

On rock seafloors, cables can be placed in shallow crevices or man-made trenches that usually prevent anchor/cable contact. Armored cables have been clad in split-pipe armor and fastened to rock seafloors with U-rods or rockbolts. These immobilization systems can usually be designed to resist the effects of loads produced by a small anchor, but if an engaging anchor does not slip loose or the anchor line does not part, then the U-rods or rockbolts may be torn out, the split-pipe broken off, and the armored cable damaged.

The probability of drag anchor contact with a cable has a moderate effect on the selection of a stabilization technique, but a large effect on the selection of a cable route.

3.2.8 Trawler Fouling

Trawling and bottom fishing are by far the greater causes of cable damage in deep water (Myers et al., 1969). In those areas where trawling is expected, cables should be buried to a depth of 2 feet (0.7 m), or, alternatively, a different cable route, free of potential trawler-induced problems, should be selected. Greater penetration depths for the new, heavier trawl equipment to be used by foreign trawlers have led to a recommended minimum burial depth of 3 feet (1 m) for cables in water depths greater than 130 feet (40 m) (NAVFAC, 1975). It is unlikely that such heavy equipment would operate in water depths shallower than 130 feet (40 m); thus, the earlier recommended 2 feet (0.7 m) required burial should prove sufficient.

3.2.9 Scouring

The potential for scour first influences the selection of a site. Then, after all considerations are weighed and the best overall site is selected, the cable route and elevation (burial depth) are designed to avoid scour problems or to adequately resist such problems.

Scour problems can be avoided by burying the cable below the maximum scour depth. In areas where rock is likely to be exposed, the cable must either be buried below the rock surface in a trench (or drill-hole) or must be fastened securely to the rock surface and protected against abrasion.

3.2.10 Ice Scoring

Normal installation and stabilization techniques are inadequate in ice-affected areas. Cables must be buried below the maximum ice-keel penetration depth expected during the cable system design life. Alternatively, on steeply sloping seafloors, it is possible that the cable could be installed in a horizontal drill-hole that passes beneath the shore and seafloor area affected by ice action (Pederson, 1974; Valent and Brackett, 1976).

3.2.11 Earth-Mass Movements (Faults)

There are two possible approaches to nearshore cable stabilization in areas of potential earth-mass movement. The first is to avoid the area. If avoidance is not possible, then the cable route should run parallel to the probable direction of travel of potential earth-mass movements. Such alignment will minimize the number of potential contacts between cable and earth-mass movements and furthermore, if a cable does get caught up, then the tension force experienced will be the minimum tension force possible under the circumstances.

3.2.12 Facilities

The existence or lack of facilities at a particular site will have little effect on the selection of a stabilization technique. However, the selection of the stabilization technique may require the addition or modification of facilities to provide adequate support for the operation. It is important to have knowledge of the facilities available while selecting and designing the stabilization system so that advantage can be taken of those that already exist and requirements for new facilities or modifications can be implemented as far in advance of the operation as possible.

3.2.13 Logistics

The logistics requirements for some stabilization techniques may make them economically unfeasible, such as when roads have to be constructed to allow large pieces of equipment to be moved to the site. In most cases, however, the logistics parameters are used to generate requirements and produce cost estimates during the planning stages. Logistics may become a critical parameter for sites with a very short weather window, where any delays due to late arrival of equipment or

supplies would jeopardize the successful completion of the operation. Good logistics planning and execution is one of the most important factors in successfully completing any operation on time and within budget.

3.2.14 Design Life

This parameter will affect both the selection of the stabilization techniques and the design of the protection system. Short-duration installations (less than 5 years) will not require as extensive an immobilization system as longer life systems. The type and amount of armor can be affected by this parameter. The system life requirements will also establish the design criteria for storm-generated waves and swell. The design engineer must be careful that the cost of the selected protection system does not exceed the anticipated cost of repair or replacement over the life of noncritical installations.

3.2.15 Criticalness of System

The required reliability will primarily affect the system design by influencing both the quality of the components used in the installation and the factor of safety specified for the design calculations. In the extreme case, more than one stabilization technique may be required to provide redundancy.

3.2.16 Type of Cable

The type of cable required for the installation will have minimal effect on the selection of the stabilization techniques, but it will be a significant factor in the design of the protection system. The diameter and density of the cable will determine its susceptibility to damage by hydrodynamic forces, as well as its ability to bury itself in sandy seafloors. The minimum bend radius and weight will dictate if special handling problems will be encountered. In some instances, the selection of a particular stabilization technique may allow for modification of the design of the cable (i.e., the use of a drilled hole or deep trench would allow the amount of armor wire to be reduced).

3.2.17 Number of Cables

The number of cables has little effect on the selection of a stabilization technique; it will, however, determine the amount of stabilization required and the extent of the installation operation. As the number of cables increases, some stabilization techniques (e.g., drilled hole and dredging) become more economically attractive since more than one cable can be protected without reproducing the protection system.

3.2.18 Length of Protected Cable

The length of cable requiring protection will have more of an economic effect rather than a technical effect on the selection of a stabilization technique. In some cases, however, the length of cable to be protected will be so great that certain techniques will be eliminated because they cannot be completed within the anticipated period of good weather (weather window). This will have to be determined after the conditions at the specific site are investigated.

3.2.19 Corridor Width

The corridor width requirements will have a negligible effect on the selection of the stabilization technique. It will, however, influence the route selection, stabilization design, and selection of installation method. In a few cases, the selected stabilization technique will govern the corridor width requirements (e.g., when large pieces of equipment, such as mechanical trenchers or tracked drills, are used).

3.2.20 Depth of Burial

The requirement to bury cables below the seafloor surface will have considerable effect on the selection of a stabilization technique. In these instances, only those techniques discussed in Section 4.5, "Burial," will be applicable. A further reduction in these acceptable techniques will be made after the type of bottom material is identified. If burial is not required due to some specific hazard, these techniques may still be found to provide the most economical means of protection.

3.3 PRELIMINARY SELECTION OF FEASIBLE CABLE PROTECTION TECHNIQUES

Several of the parameters discussed in the previous section influence the feasibility of implementing the various cable protection techniques at a given site. The parameters most often found to influence the preliminary selection (or feasibility) of a cable protection system include:

(1) Bottom material

(2) Topography
   - (a) Seafloor
   - (b) Beach

(3) Underwater visibility

(4) Minimum wave height during installation

(5) Minimum current velocity during installation

(6) Minimum wind velocity during installation

(7) Potential hazards to the system

Table 3-1 assesses the suitability of each protection technique for several ranges of conditions that would normally be found for the particular parameter under consideration. In the rating of the protection techniques, six feasibility classifications were established. The notation used and the definition of these classifications are presented in Table 3-1. Footnotes have also been added where additional clarification or discussion was required. To use Table 3-1, one selects the appropriate description or range for each major parameter and then reads the evaluation for each protection technique. A feasibility rating work sheet (Table 3-2) has also been provided for tabulating the total feasibility for each potential site.

Because of the range of values of each parameter that is possible at any site and the interaction or influence of parameters on each other, Table 3-1 should be used as a guide to the initial selection or screening of the various techniques rather than an absolute indicator of their feasibility.

Table 3-1. Suitability of Different Protection Techniques for Various Important F

Parameters	Mass Anchor Systems										Tie-Downs	
		Split-Pipe					Concrete					
	Armor Wire	Diver Applied	Shore Applied	Under-Running	Overhaul Cable	Pipe Casing	Sacked	Cast-In-Place	Precast Elements	Chain	Pins	Grouted Fasteners
BOTTOM MATERIAL												
• Silty/clay (mud)	YES	NO	YES	YES	NO	YES	NO	NO	NO	NO	YES	NO
• Stiff silty clay	YES	N.R.	YES	YES	N.R.	YES	N.R.	N.R.	N.R.	YES	YES	NO
• Sand	YES	YES	YES	YES	YES	YES	NO	NO	NO	YES	YES	NO
• Loose gravel, cobble, boulders	YES	YES	YES[c]	YES	YES[c]	YES	YES	YES	YES	YES	NO	NO
• Hard rock	YES	YES	YES[c]	YES	YES[c]	YES	YES	YES	YES	YES	NO	YES
• Coral and most soft rock	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	NO	YES
SEAFLOOR TOPOGRAPHY												
• Continuous or gently changing contours	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	LIM[j]	LIM[j]
• Irregular contours; 2- to 5-ft discontinuities	YES[k]	YES	YES[c]	YES	YES[c]	LIM[m]	YES	YES	YES	YES	NO[j]	YES[j]
• Rugged contours; 5-ft discontinuities; numerous suspensions	YES[k]	YES	YES[c]	YES	YES[c]	LIM[m]	YES	YES	YES[n]	YES	NO[j]	YES[j]
BEACH TOPOGRAPHY												
• Steep bluff with little or no beach; difficult access from land	N.S.	YES	NO	YES	NO	NO	N.S.	N.S.	N.S.	YES	N.S.	N.S.
• Short beach; good access from land	N.S.	YES	NO	YES	YES	NO	N.S.	N.S.	N.S.	YES	N.S.	N.S.
• Deep beach with good access from land	N.S.	YES	YES	YES	YES	YES	N.S.	N.S.	N.S.	YES	N.S.	N.S.
UNDERWATER VISIBILITY												
• Less than 1 foot	N.S.	N.R.	YES	YES	N.R.	YES[t]	N.R.	N.R.	N.R.	N.R.	N.R.	NO
• 1 to 5 feet	N.S.	LIM	YES	YES	LIM	YES[t]	N.R.	LIM	LIM	LIM	N.R.	N.R.
• 5 to 15 feet	N.S.	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES
• Greater than 15 feet	N.S.	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES
MINIMUM WAVE HEIGHT (during installation)												
• Less than 2 feet	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES
• 2 to 6 feet	YES	LIM[r]	YES	LIM[s]	LIM[r]	YES	LIM[r]	N.R.[s]	LIM[s]	LIM[r]	LIM[r]	LIM[r]
• Greater than 6 feet	N.R.[u]	N.R.[r]	N.R.[u]	N.R.[s]	N.R.[r]	N.R.[u]	N.R.[r]	N.R.[s]	N.R.[s]	N.R.	N.R.	N.R.
MINIMUM CURRENT VELOCITY (during installation)												
• Less than 0.1 knot	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES
• 0.1 to 0.5 knot	YES	YES	YES	YES	YES	YES	LIM[v]	NO[v]	LIM[w]	YES	YES	YES
• 0.5 to 1.5 knots	LIM[x]	LIM[y]	LIM[x]	LIM[x]	LIM[y]	LIM[x]	NO	NO	LIM[w]	LIM[y]	LIM[y]	LIM[y]
• Greater than 1.5 knots	LIM[x]	NO	LIM[x]	LIM[x]	N.R.	LIM[x]	NO	NO	NO	NO	NO	NO
AVERAGE WIND VELOCITY (during installation)												
• Less than 10 knots	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES
• 10 to 20 knots	YES	YES	YES	LIM[z]	YES	YES	YES	LIM[z]	LIM[z]	YES	YES	YES
• Greater than 20 knots	LIM[x]	N.R.[aa]	LIM[x]	LIM[x,z]	N.R.[aa]	LIM[x]	N.R.[aa]	N.R.[aa]	N.R.[aa]	N.R.[aa]	N.R.[aa]	N.R.[aa]
POTENTIAL HAZARDS												
• Anchors	NO	LIM[bb]	LIM[bb]	LIM[bb]	LIM[bb]	LIM[bb]	LIM[bb]	LIM[bb]	LIM[bb]	LIM[bb]	LIM[bb]	LIM[bb]
• Trawlers	NO	LIM[cc]	LIM[cc]	LIM[cc]	LIM[cc]	LIM[cc]	NO	NO	NO	NO	NO	N.A.[dd]
• Scouring	YES[ee]	YES[ee]	YES[ee]	YES[ee]	YES[ee]	NO	NO	NO	NO	YES[ee]	NO	N.A.
• Ice Scoring	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO
• Earth-Mass Movements (faults)[gg]	LIM	LIM	LIM	LIM	LIM	LIM	NO	NO	NO	LIM	LIM	YES

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ility of Different Protection Techniques for Various Important Parameters

nor Systems					Tie-Downs				Burial Systems					
	Concrete													
Pipe Casing	Sacked	Cast-In-Place	Precast Elements	Chain	Pins	Grouted Fasteners	Rockbolts	Tensioning	Self Burial	Jetting	Dredging	Explosive Excavation	Mechanical Trenching	Drilled Hole
YES	NO	NO	NO	NO	YES	NO	NO	YES	LIM	YES	YES	NO	NO	NO
YES	N.R.	N.R.	N.R.	YES	YES	NO	NO	YES	NO	LIM	YES	NO	LIM	NO
YES	NO	NO	NO	YES	YES	NO	NO	YES	YES	YES[a]	YES[b]	NO	YES	NO
YES	YES	YES	YES	YES	NO	NO	NO	YES	NO	NO	LIM	LIM	LIM[d]	LIM
YES	YES	YES	YES	YES	NO	YES	YES	YES	NO	NO	NO	YES	YES[e]	YES[f]
YES	YES	YES	YES	YES	NO	YES	YES[g]	YES	NO	NO	YES	YES[h]	YES	YES[i]
YES	YES	YES	YES	YES	LIM[j]	LIM[j]	LIM[j]	YES	LIM[j]	LIM[j]	YES	LIM[j]	YES	NO
LIM[m]	YES	YES	YES	YES	NO[j]	YES[j]	YES	YES	NO	NO	LIM[j]	YES	YES	LIM
LIM[m]	YES	YES	YES[n]	YES	NO[j]	YES[j]	YES	YES	NO	NO	NO	YES	NO	YES
NO	N.S.	N.S.	N.S.	YES	N.S.	N.S.	N.S.	N.R.	LIM[o]	LIM[o]	YES	YES	N.R.	YES[p]
NO	N.S.	N.S.	N.S.	YES	N.S.	N.S.	N.S.	YES	YES	YES	YES	YES	YES	YES
YES	N.S.	N.S.	N.S.	YES	N.S.	N.S.	N.S.	YES	YES	YES	YES	LIM[o]	YES	NO[q]
YES	N.R.	N.R.	N.R.	N.R.	N.R.	NO	NO	N.S.	N.S.	N.R.	N.S.	NO	NO	N.S.
YES	N.R.	LIM	LIM	LIM	N.R.	N.R.	N.R.	N.S.	N.S.	N.R.	N.S.	N.R.	NO	N.S.
YES	YES	YES	YES	YES	YES	YES	YES	N.S.	N.S.	YES	N.S.	YES	N.R.	N.S.
YES	YES	YES	YES	YES	YES	YES	YES	N.S.	N.S.	YES	N.S.	YES	YES	N.S.
YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES
YES	LIM[r]	N.R.[s]	LIM[s]	LIM[r]	LIM[r]	LIM[r]	LIM[r]	YES	YES	LIM[r]	LIM[t]	LIM[r]	LIM[t]	YES
N.R.[u]	N.R.[r]	N.R.[s]	N.R.[s]	N.R.	N.R.	N.R.	N.R.	N.R.[s]	LIM[u]	N.R.	N.R.[s]	N.R.[r]	LIM[t]	YES
YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	N.S.
YES	LIM[v]	NO[v]	LIM[w]	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	N.S.
LIM[x]	NO	NO	LIM[w]	LIM[y]	LIM[y]	LIM[y]	LIM[y]	LIM[x]	YES	LIM[y]	YES	LIM[y]	LIM[t]	N.S.
LIM[x]	NO	NO	NO	NO	NO	NO	NO	LIM[x]	YES	NO	N.R.	NO	LIM[t]	N.S.
YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	YES	N.S.
YES	YES	LIM[z]	LIM[z]	YES	YES	YES	YES	YES	YES	YES	LIM[z]	YES	YES	N.S.
LIM[x]	N.R.[aa]	N.R.[aa]	N.R.[aa]	N.R.[aa]	N.R.[aa]	N.R.[aa]	N.R.[aa]	LIM[x]	YES	N.R.[aa]	N.R.[z]	N.R.[aa]	LIM[aa]	N.S.
LIM[bb]	LIM[bb]	LIM[bb]	LIM[bb]	LIM[bb]	LIM[bb]	LIM[bb]	LIM[bb]	NO	NO	YES	YES	YES	YES	YES
LIM[cc]	NO	NO	NO	NO	NO	N.A.[dd]	N.A.[dd]	NO	YES	YES	YES	YES	YES	YES
NO	NO	NO	NO	YES[ee]	NO	N.A.	N.A.	NO	YES[ee]	LIM[ff]	LIM[ff]	N.A.	LIM[ff]	YES
NO	NO	NO	NO	NO	NO	NO	NO	NO	NO	LIM[ff]	LIM[ff]	YES	YES	YES
LIM	NO	NO	NO	LIM	LIM	YES	YES	NO	LIM	LIM	LIM	LIM	LIM	LIM

NOTATION:

YES — Technique is feasible.

NO — Technique is technically unfeasible for the conditions found at the site.

LIM — Limited feasibility. Depends on other parameters, further definition of conditions at the site, or may be extremely sensitive to the range of that parameter.

N.R. — Not recommended for this application. These techniques may be technically feasible, but have shown poor performance for previous installations due to: (a) premature failure of the system, (b) difficulty in implementing or installing the system with present tools and technology, (c) hazardous conditions that may be created for personnel when coupled with other conditions normally encountered with or resulting from this value of the parameter, or (d) poor economic choice.

N.A. — Not applicable.

N.S. — Technique is feasible and not sensitive to this parameter.

FOOTNOTES:

[a] Jetting alone is not efficient in sand; jetting combined with a plow and stinger is much better.

[b] Inefficient unless cable is placed immediately behind the dredge.

[c] Boulders or rugged topography in shallow water may prevent cable or pipe from being dragged out to sea.

[d] Success variable; mechanical difficulties will be frequent.

[e] Very slow advance rate, but finished product is totally protected.

[f] Economically drilled hole lengths are 1,000 to 2,000 meters; usually cost effective only when all else fails.

[g] Special large-headed rockbolts are generally required in coral.

[h] Yield in coral is highly variable.

[i] Depends on slope of seafloor and presence of buried sand pockets.

[j] Depends on seafloor material.

[k] Generally not sufficient by *itself* in these environments.

[m] Feasibility depends on length of suspended section.

[n] Steep slopes may cause large precast elements to become unstable.

[o] May prove impossible to accomplish because seafloor material near bluff will not be suitable for cable burial.

[p] Drilled hole would be started from top of bluff.

[q] Drill-hole not technically feasible for long traverses.

[r] May be difficult *or dangerous* to conduct diving operations; actual effect on diver performance will depend on water depth and period of waves.

[s] May be difficult *or dangerous* for barges, cable ships, and diving support craft to operate close to shore.

[t] Depends on configuration of the trenching equipment and whether it is remotely controlled (from beach) or diver-operated.

[u] Difficult *or dangerous* for personnel to remove buoyancy from the cable and properly position it on the seafloor.

[v] Excessive currents may take the cement out of the concrete mix before it can set.

[w] Depends on configuration of elements; diver placement may be difficult or impossible due to excessive drag forces.

[x] May create large catenaries in the cable that cause excessive loads or result in positioning far from the desired cable route.

[y] Productivity of divers decreases rapidly as current increases in this range.

[z] May be difficult to keep surface support ship on station without the use of an elaborate mooring system.

[aa] Extremely difficult to conduct safe diving operations.

[bb] Depends on size of anchor and ship and type of bottom material.

[cc] On a sand seafloor with no exposed rock, the cable/pipe system will normally sink below the depth of penetration of the otter boards.

[dd] Trawling is not usually conducted in rocky seafloor areas where these techniques are applicable.

[ee] Assumes that sufficient slack is provided so that cable remains on the new bottom.

[ff] Depends on the magnitude of change of seafloor elevation anticipated and burial depth capabilities of available equipment.

[gg] Faulting is not usually a significant problem with cables; landslides initiated by faulting, however, will normally overstress and break cables that are not laid parallel to the direction of slide movement.

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Table 3-2. Feasibility Rating Work Sheet for Each Potential Cable Route

Parameter	Mass Anchor Systems										Tie-Downs				Burial Systems					
		Split-Pipe					Concrete													
	Armor Wire	Diver Applied	Shore Applied	Under-Running	Overhaul Cable	Pipe Casing	Sacked	Cast-in-Place	Precast Elements	Chain	Pins	Grouted Fasteners	Rock Bolts	Tensioning	Self Burial	Jetting	Dredging	Explosive Excavation	Mechanical Trenching	Drilled Hole
Bottom Material																				
Seafloor Topography																				
Beach Topography																				
Underwater Visibility																				
Wave Height (during installation)																				
Current Velocity (during installation)																				
Wind Velocity (during installation)																				
Potential Hazards																				
FEASIBILITY RATING																				

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Chapter 4

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Chapter 4

CABLE PROTECTION TECHNIQUES

4.1 INTRODUCTION

This chapter presents a discussion of 15 individual cable protection techniques. Included are a description of major components and equipment; estimates on manpower requirements and production rates; and, when available, any data which relate to the design and installation of the system. Also included is a detailed evaluation of the effect that each of the parameters presented in Chapter 2 has on the feasibility of the techniques. This section is intended to provide a cross check for the preliminary screening process and to allow a more detailed assessment where some doubt about the technique applicability may have existed.

The techniques presented in this chapter have been separated into four main groups, according to the type of protection provided: (1) mass anchors, (2) tie-downs, (3) burial, and (4) tensioning. The preliminary screening list obtained from Chapter 3 is intended as a guide to the applicable sections of this chapter.

This chapter provides a basis for thorough understanding of the various feasible protection techniques before the actual system is designed.

4.2 MASS ANCHORS

Mass anchor stabilization techniques, by virtue of their weight and the resulting friction between the seafloor and the system components, allow the cable to resist environmental hazards. To the extent that the mass anchor succeeds in resisting environmental influences (e.g., hydrodynamic and anchor drag forces), it can be considered a protection system. When, however, the friction forces are exceeded by the environmental influences, the mass anchor becomes part of the cable system and may itself require additional protection.

The techniques presented in this section include: (1) armor wire, (2) split pipe, (3) oil field pipe, (4) concrete, and (5) chain.

4.2.1 Armor Wire

Background. Armor wires on nearshore ocean cables must provide the necessary strength to resist the forces applied to the cable during laying and must protect the cable core after installation (Myers et al., 1969).

The use of armoring as the only stabilization means is not common except in calm water (lakes and lagoons) where cable hazards are almost nonexistent. Armoring may be sufficient beyond the surfzone at some ocean sites where the seafloor is composed of a deep sand layer, thick enough to prevent rock exposure. At these sites, cables with adequate armor weighting will tend to bury themselves due to wave action (see Section 4.4.1). Even if alternative stabilization techniques are selected, the nearshore cable will usually be armored to resist the loads produced in laying and to provide interim protection until the protection system is completely installed.

Description. Armor wires are usually comprised of galvanized steel wires of various sizes and tensile strengths. Individual wires are usually coated with a tar compound or jacketed with plastic or neoprene compound for added corrosion protection.

At sites with severe ocean environments or with exposed rock on the seafloor, the shore end cable will normally be protected by two layers of heavy armor wires. These "double-armored" shore-end cables always have the two sheathings applied in the same direction of lay. In some cases, to provide more abrasion protection, the outer armor sheathing is applied with a higher lay angle (short lay) than the inner armor (Simplex Manual).

Each cable manufacturer appears to have adopted a different method of identifying submarine cables and armor wire configurations. Table 4-1 gives the designation and sizes of the most commonly used armor; Table 4-2 indicates some of the many ways that manufacturers identify cables.

Procedure. The procedure for applying armor wire to ocean cables is beyond the scope of this handbook. If information on this process is required, the cable manufacturer should be contacted.

Table 4-1. External Armor Wire Types for Ocean Cables[a]

Designation	Armor BWG	No. of Layers	Diameter (in.)	Comments
AA (Heavy Shore)	1	2	0.300	Galvanized mild steel
A (Light Shore)	1	1	0.300	Galvanized mild steel
AJAJ or JJ	6	2	0.203	Galvanized mild steel, each armor wire Neoprene jacketed to 0.300-in. OD
E	4	1	0.238	Continental Shelf use and protected landings; not common
B_6	6	1	0.203	Common Continental Shelf cable armor
B_8	8	1	0.165	Common Continental Shelf cable armor
D (Deep Sea)	13-1/2	1	0.086	High tensile on order of 250k psi

[a]From G. D. Cullison, 1975.

Table 4-2. Examples of Designations of Ocean Cables[a]

Designation	Description
LPANY 30/1	Western Electric Company (WECO) designation LP (coaxial) type cable with type A armor, new yarn outer serving, 30 No. 1 BWG armor wires.
21QAAOY 38/1 30/1	21 Quad (84 conductor) cable with type AA armor, old yarn outer serving, 38 No. 1 BWG wires in outer armor layer, 30 No. 1 BWG wires in inner armor layer.
21QDNNY 60/.112	21 Quad cable with type D armor, new nylon yarn outer serving, 60 0.112-in.-diam armor wires.
SDL4	AT&T designated SD type coaxial cable with one layer of 17 0.203-in.-diam armor wires, each jacketed to 0.340-in. OD.
LPJJ	WECO designation LP type cable with two layers of 0.203-in.-diam armor wires, each jacketed with neoprene to 0.300-in. diam, 38 armor wires in outer layer, 30 armor wires in inner layer.
.160"/.620" 24/.086"	Simplex Wire and Cable Co. designations for a coaxial cable with 0.160-in.-diam inner conductor , 0.620-in.-diam over the inner insulation with 24 0.086-in.-diam extra high strength steel armor wires. The overall diameter is 1.25 in., although that is not mentioned in the specification designation.

[a]From G. D. Cullison, 1975.

Design Considerations. The bulk density of the armored cable will determine if self-burial will occur. A minimum density of 119 lb/ft^3 (1.9 g/cm^3) is required to cause sinkage in sand, silts, and soft clays when sufficient wave action is present. Table 4-3 lists properties of typical nearshore cables.

Table 4-3. Properties of Typical Nearshore Ocean Cables

Designation	Description	Weight (lb/ft)		Bulk Density		Breaking Strength (lb x 1,000)
		Air	Water	g/cm^3	lb/ft^3	
SDL 3 (List 3)	Coaxial cable with 15 armor wires 0.203-in.-diam jacketed with neoprene to 0.300-in. diam	5.27	3.56	3.10	194	56.4
SDL 4 (List 4)	Coaxial cable with 17 armor wires 0.203-in.-diam jacketed with neoprene to 0.340-in. diam	7.28	5.27	3.62	226	70.6
SDL 5 (List 5)	Coaxial cable 2 with two armor sheaths, 17 wires 0.203-in.-diam jacketed to 0.340-in. diam, and 23 wires 0.203-in.-diam jacketed to 0.340-in. diam	14.75	11.45	4.48	280	89.5

The minimum breaking strength of ocean cables is based strictly on the strength of the armor wires. The remaining materials used in cable construction add little to the breaking strength of the cable. For double-armored cables with a short lay on the outer armor sheathing, only the inner armor wires are used to calculate the breaking strength. The actual breaking strength is somewhat greater but usually cannot be predicted with any degree of accuracy (Simplex Manual).

If the breaking strength and number of armor wires is not available from the manufacturer, an estimation may be made from the equation:

$$F_B = \left[\frac{\pi}{d_w}(D_c + d_w)\right]\left[\frac{\pi}{4}\sigma_u \bar{d}_w^2\right] \qquad (4\text{-}1)$$

where F_B = minimum breaking strength (lb)

d_w = armor wire diameter including jacket (in.)

D_c = core diameter of cable (in.)

$\bar{d}_w$ = armor wire diameter without jacket (in.)

σ_u = ultimate stress of armor wire material (lb/in.^{2})

When the armor is not individually jacketed, then $d_w = \bar{d}_w$ and Equation 4-1 reduces to:

$$F_B = \frac{\pi^2}{4} \sigma_u (D_c + d_w) d_w \tag{4-2}$$

If the armor wire gage size is known, then Table 4-4 may be used to estimate the weight and breaking strength of the cable.

Table 4-4. Steel Armor Wire[a]

BWG Size	Diameter (in.)	Approximate Weight (lb/1,000 ft)		Approximate Breaking Strength[b] (lb)
		Air	In Seawater	
0	0.340	313	272	9,079
1	0.300	244	212	7,068
2	0.284	218	189	6,335
3	0.259	182	158	5,268
4	0.238	153	133	4,449
5	0.220	131	114	3,801
6	0.203	112	97	3,237
7	0.180	87	76	2,545
8	0.165	74	64	2,138
9	0.148	60	52	1,720
10	0.134	49	43	1,410
11	0.120	39	34	1,131
–	0.112	33	29	985
12	0.109	32	28	933
13	0.095	25	22	709
–	0.086	20	17	580
14	0.083	19	16	541
15	0.072	14	12	407
16	0.065	11	10	332

[a] From Simplex Manual for Submarine Cables.

[b] Based on 100,000 psi.

Selection Factors.

BOTTOM MATERIAL AND TOPOGRAPHY. Armoring, as the only means of stabilization, is only practical in areas with sandy, silty, or clay seafloors where self-burial is likely to occur and where the probability of encountering hazards is minimal. On rocky seafloors, not only will armor be required for initial abrasion protection, but additional stabilization techniques will also be required.

WAVES. In areas where self-burial will occur, waves are beneficial in accelerating the burial process. Extremely large waves >10 feet (>3 meters) will adversely affect the installation process, and landing* a cable under these conditions should be avoided.

CURRENT. The current, along with waves, produces hydrodynamic forces, which the stabilization system must resist. If self-burial occurs, the cable is removed from influence of these forces. During installation, longshore currents will tend to displace the cable from the preselected route by

*Attaching a cable to shore.

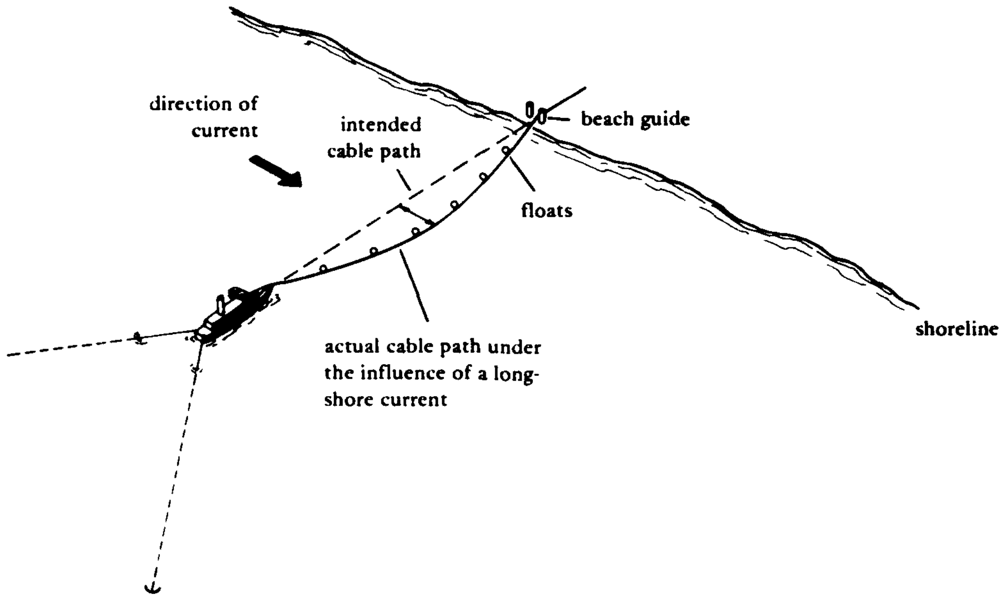

Figure 4-1. Effect of longshore current on cable path.

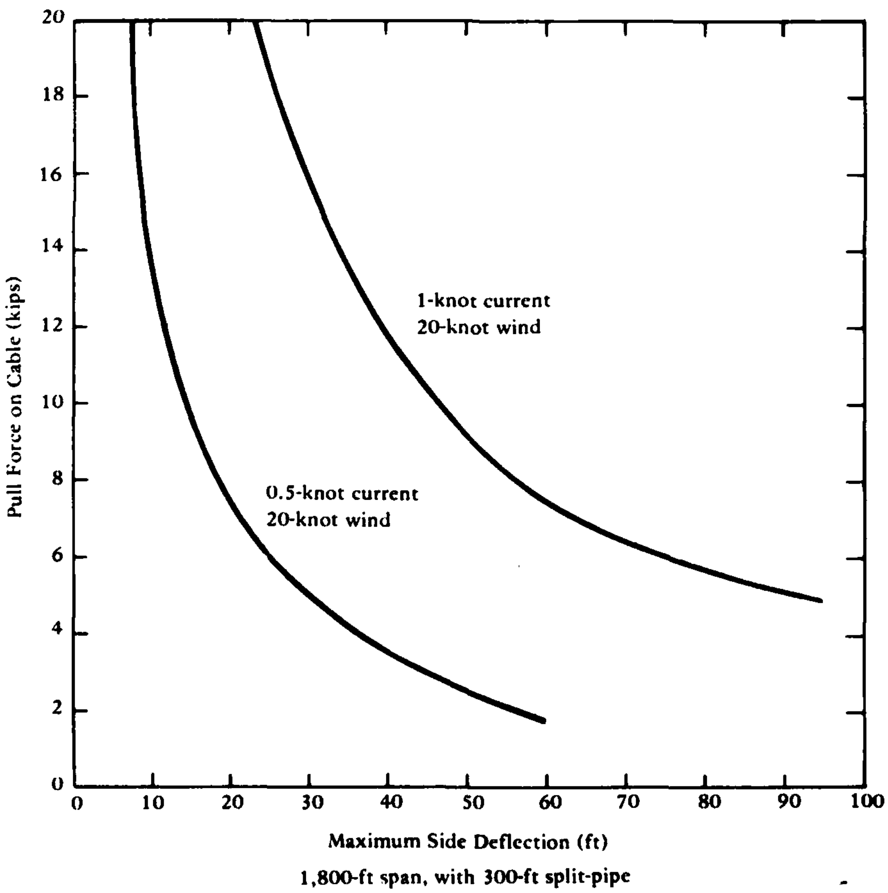

Figure 4-2. Maximum deflection versus cable tension for cables exposed to lateral wind and current loads (from: Project Execution Plan FPO-1-77(15)).

inducing a catenary shape to the cable (Figure 4-1). To keep the cable within the specified corridor (Section 2.6.6), a tensile force must be applied to the cable to reduce deflection to an acceptable distance. Figure 4-2 shows the fore-deflection relationship for a specific case of 1800 feet of SDL5 cable with 300 feet of split-pipe supported by float ballons.

LOGISTICS. No specialized support is required other than that necessary to lay the cable. Each additional layer of armor wire will affect shipping costs and cable handling equipment because of the increase in both the weight and minimum bend radius.

WEATHER WINDOW. A 4- to 8-hour weather window is usually sufficient for this technique since the nearshore portion of most cable installations can be laid within this time with good weather conditions. Beach preparation may require 2 to 6 weeks of good weather, depending on the extent of the construction to be done.

VISIBILITY. Underwater visibility has no effect on the selection of this technique for stabilization.

HAZARDS. If any of the hazards discussed in Section 2.4 has a moderate to high probability of occurring at the site, armoring as the only form of stabilization will not be adequate.

WIND. During the cable landing phase of the installation, wind blowing perpendicular to the cable path will have the same effect as a longshore current, causing cable displacement from the preselected route.

DESIGN LIFE. For cable systems with long operational life requirements ($\approx$20 years), the use of armoring as the only stabilization means is questionable because of the increasing probability of hazard occurrences (i.e., anchor drag, trawler activity, etc.).

LENGTH OF PROTECTED CABLE. The length of cable requiring armor protection has no effect on whether or not this technique should be specified.

4.2.2 Split-Pipe

Background. One of the most common techniques for stabilizing shore-end cables through the surf zone and over rocky bottoms is by the use of heavy nodular cast-iron half-pipe sections, commonly referred to as "split-pipe" (Figure 4-3). Normally, split-pipe is applied over the cable out to a depth where hydrodynamic forces are no longer of significance. Although its high in-place cost necessitates split-pipe used only in critical areas, it is used to some extent on most ocean cable shore-ends.

Split-pipe performs two basic functions: (1) provides abrasion protection for the cable, thus increases cable resistance to chafing; and (2) increases the system density, and, therefore, decreases the cable

system's sensitivity to hydrodynamic forces. The first function is performed extremely well when the cable system passes over a rocky bottom. The use of split-pipe for the sole purpose of increasing cable system density to promote self-burial is of questionable value, since the density is normally increased only by a factor of about 2; and according to Van Daalen and Van Steveninck, 1970, armored cables already have a bulk density high enough for self-burial (119 lb/ft^3 - 1.9 g/cm^3 - or greater ensures self-burial). This has been verified by experience. Therefore, cables are protected with split-pipe primarily for chafing resistance, and the gains in increased system density are simply an additional benefit.

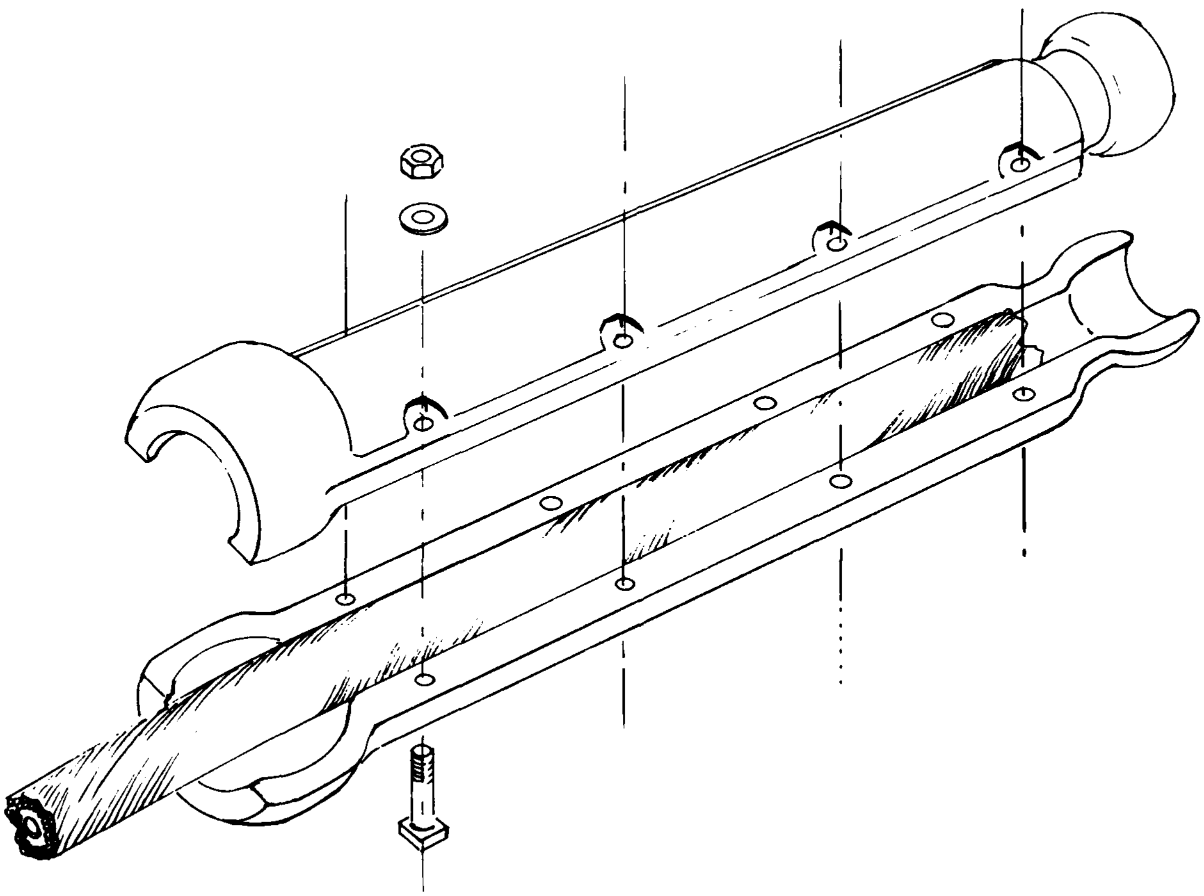

Figure 4-3. Split pipe.

Commercial data are unavailable; therefore, the information in this section of the handbook is based only on documented, post-1970 Navy applications. Users are urged to provide the Chesapeake Division of the Naval Facilities Engineering Command (FPO-1) with updated data that either amplifies or is in disagreement with information provided herein.

Description. Split-pipe comes in 39-inch-long half-pipe sections weighing 75 pounds per half section dry and 64 pounds wet for the 3-1/2-in. diam pipe. Figure 4-4 and Table 4-5 provide information about the most commonly used type of split pipe. Bolted together half sections of split pipe produce a ball-in-socket connection arrangement with an assembled length of 36 in. per set of half-sections. Standard split-pipe has a minimum internal diameter of 3-1/2 inches, and special large bore split-pipe is fabricated with a minimum internal diameter of 5 inches (Figure 4-5). This pipe is used for special applications like multiple cable runs, large quad cables, and coverings for spliced sections of cable.

Each ball-in-socket connection will accommodate nominally 15 degrees of articulation (Figure 4-6) from the longitudinal axis, producing a 12-foot minimum turning radius. However, randomly selected sections of 3-1/2-in. ID split-pipe have been noted as having only 12 degrees of articulation.

The pipe is quite tough and will withstand 70,000-pound axial loads (if high-grade fasteners are used). Table 4-6 lists split-pipe failure loads for various fastener types. At this time no known data are available on split-pipe's ability to withstand bending loads applied to the ball-in-socket joint. Therefore, until such data are available, installation techniques should always limit the radius of piped cable to 12 feet or greater.

Each section (two half-sections) of split-pipe utilizes eight fasteners to secure the half-sections together. The most common fastener in use at this time is a 5/8-inch stainless steel bolt, lock washer, and nut assembly (some elastic stop nuts have been used). Mild steel fasteners have also been used, but removal for cable repairs after they have corroded is difficult and time-consuming. Various blind fastening bolt techniques are under study. These fasteners (Figure 4-7) show promise in reducing installation time, preventing loosening from vibration and reducing the number of personnel required for installation. At this time, however, they are still being tested to determine their service life in the submarine environment, and techniques for removing them from split-pipe are not well-established.

The two fasteners closest to the bell-end of the split-pipe provide the strength at the ball-in-socket connection. Additional fasteners supply additional clamping strength and redundancy to the bell-adjacent fasteners (with greatly reduced strength, however) (Brackett and Tausig, 1977b). If, for some reason, time becomes extremely critical during application, the on-site technical authority would have some justification to utilize only two fasteners nearest the bell-end for each assembled split-pipe section.

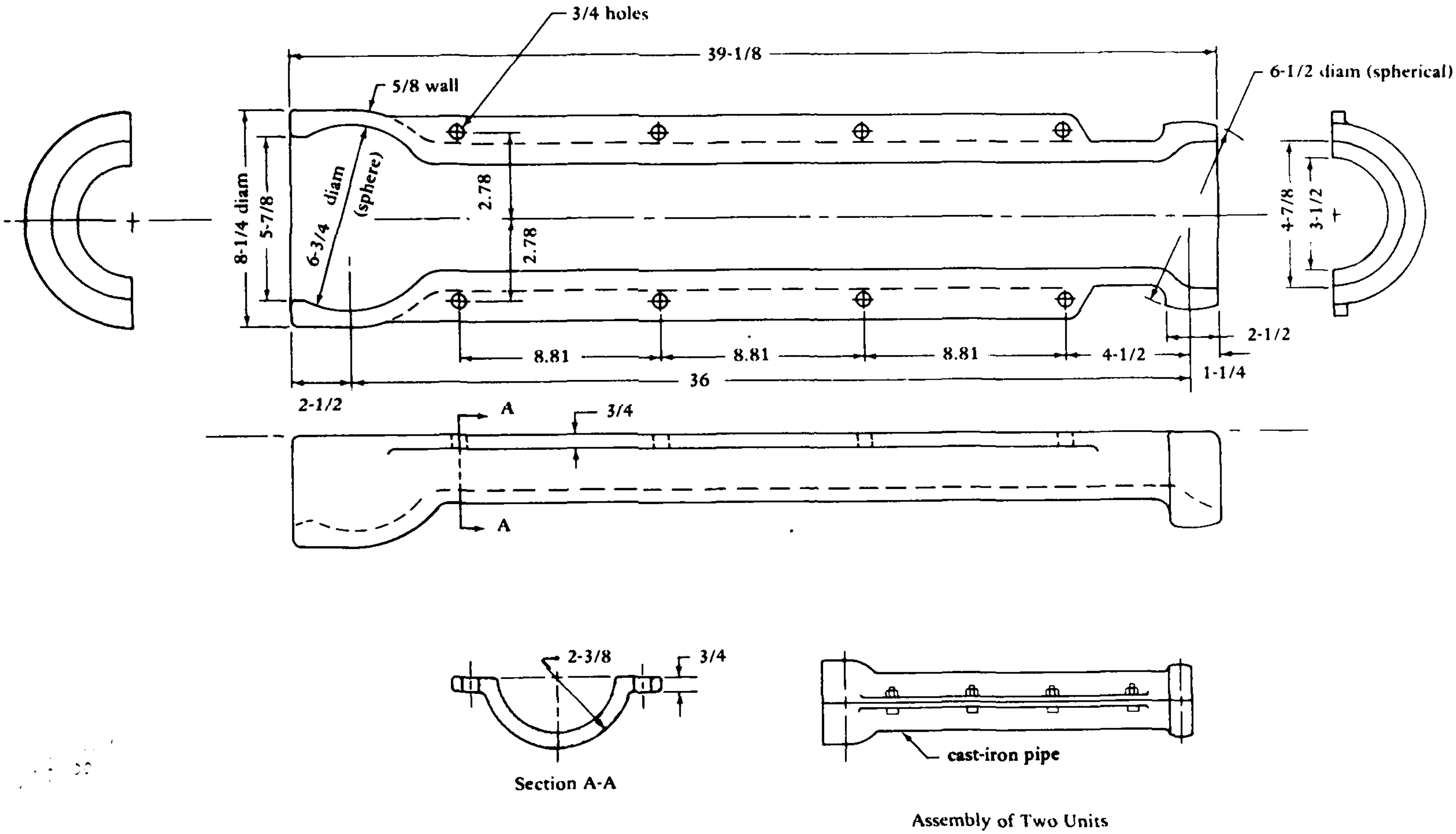

Figure 4-4. Split-pipe, 3-1/2 in. diameter.

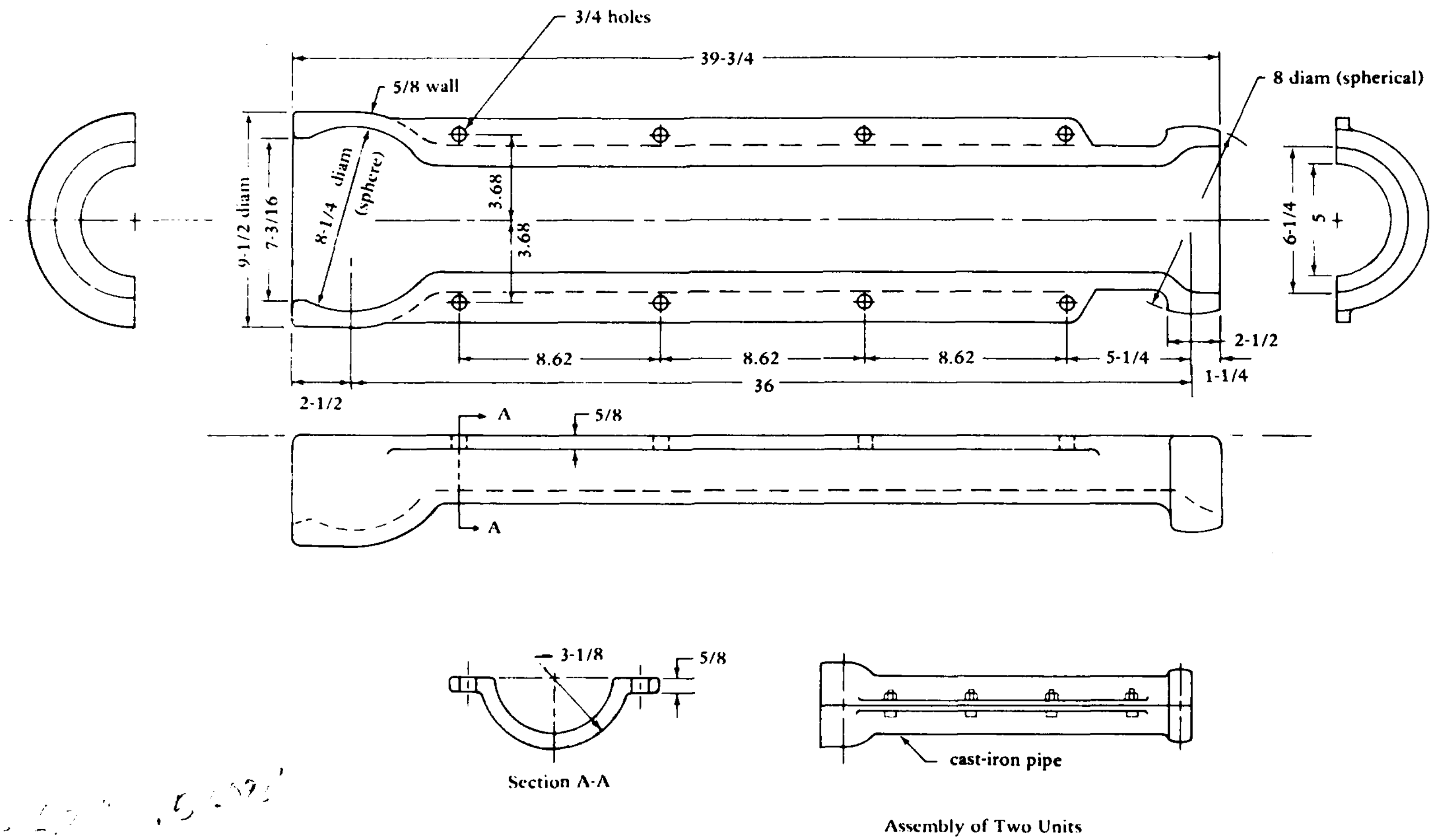

Figure 4-5. Split-pipe, 5-in. diameter.

Table 4-5. Description of Split-Pipe[a]

Description	Measurement
Length of Section When Assembled, ft	3
Cost Estimate (3-1/2-in-diam pipe in 1975), \$/ft	30
Tensile Failure (bell-separated), lbf	70,000
Recommended Safe Working Load, lbf	35,000
Beach Pulling Load on Sand, lbf/ft	30
Beach Pulling Load in 3 ft of Water (on sand), lbf/ft	34
Tensile Strength of Cast-Iron Material, psi	22,000
Failure Modes:	
Split-Pipe Bell, lbf yield	40,000
lbf ultimate	60,000
Bell Separation, lbf	70,000
Split-Pipe Flange, lbf yield	55,000
lbf ultimate	88,000
Boltholes Elongated, lbf	60,000
3-1/2-In.-Diam Pipe Weight (one-half section):	
In air, lbf/ft	21.5
In seawater, lbf/ft	20.0
5-In.-Diam Pipe Weight (one-half section):	
In air, lbf/ft	30.2
In seawater, lbf/ft	28.6

[a] Data from Thibeaux, 1972.

Eight fasteners are required for each secton of split-pipe (every 3 feet). For every 100 feet of split-pipe 267 fasteners are needed. It is suggested that approximately 300 fasteners per 100 feet be supplied to account for the 10% losses expected from defective fasteners.

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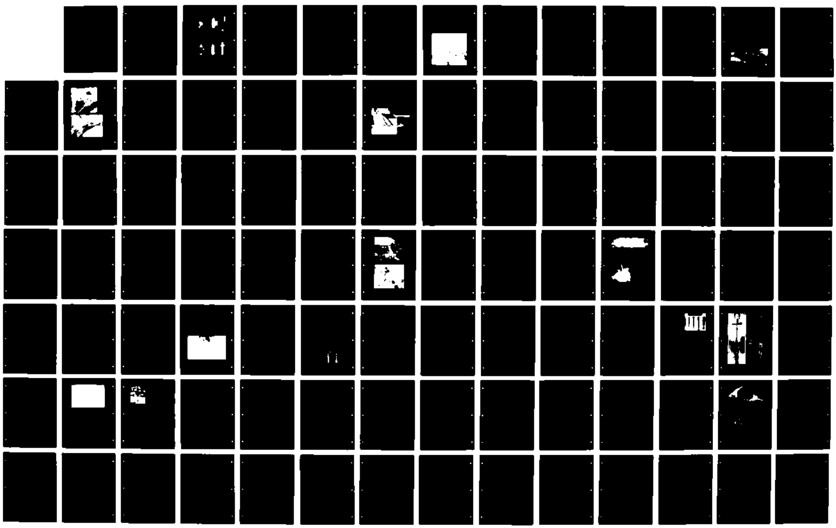

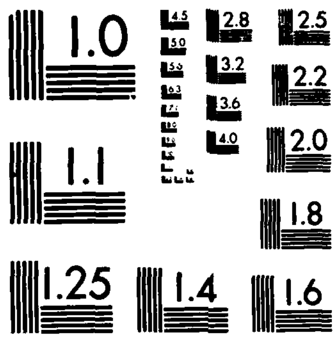
1.0
4.5
5.0
5.6
6.3
2.8
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1.25
1.4
1.6

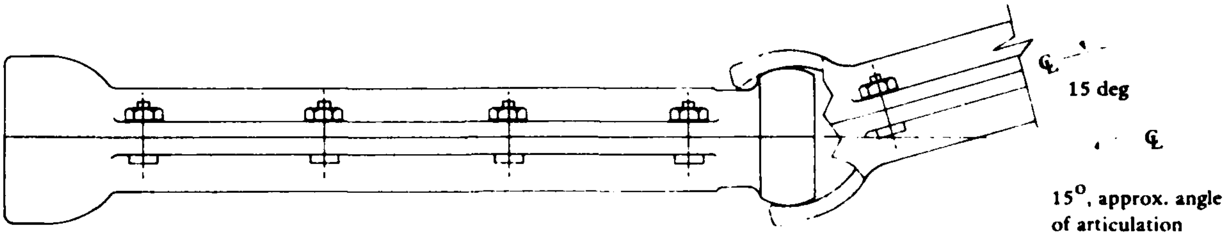

Figure 4-6. Ball-in-socket joint.

Installation Techniques. In recent ars, four different split-pipe installation techniques have been utilizea by the Navy. These techniques are: (1) applying the split-pipe to the cable by divers (cable already resting on the seafloor); (2) applying the split-pipe to the cable on the beach and then dragging the piped cable to sea; (3) applying the split-pipe to a floating cable from an under-running vessel; and (4) applying short lengths of split-pipe to the cable on the beach and dragging the split-pipe over the cable to sea. Each of these techniques is appropriate at different times and a discussion of what technique best fits a given situation follows.

Until about 1975, the conventional approach to applying split-pipe was to have divers assemble it after the cable had been laid on the bottom in its desired location. This technique requires considerably more time than the other approaches and, of course, requires more diving services. The technique is straightforward in ideal conditions but becomes more difficult and approaches the impossible as sea conditions become more inhospitable (e.g., reduced visibility, large breaking surf, or increasing water depth).

Because of the expense of diver services, limited available operating time, and the occasional severe underwater conditions found at some cable locations, other techniques have been developed. Techniques (2) and

Table 4-6. Split-Pipe/Fastener Pull Test[a]

Fastener	Tensile Load of Pipe at Failure (lb)
Blind Bolt – Huck BOM	72,400
Stainless Steel Nut and Bolt	70,000
Carbon Steel Nut and Bolt	62,000
Blind Bolt – Hishear	
Mild Steel	53,100
Stainless	48,500
PVC Nut and Bolt	16,800

[a] Data from Brackett and Tausig (1977).

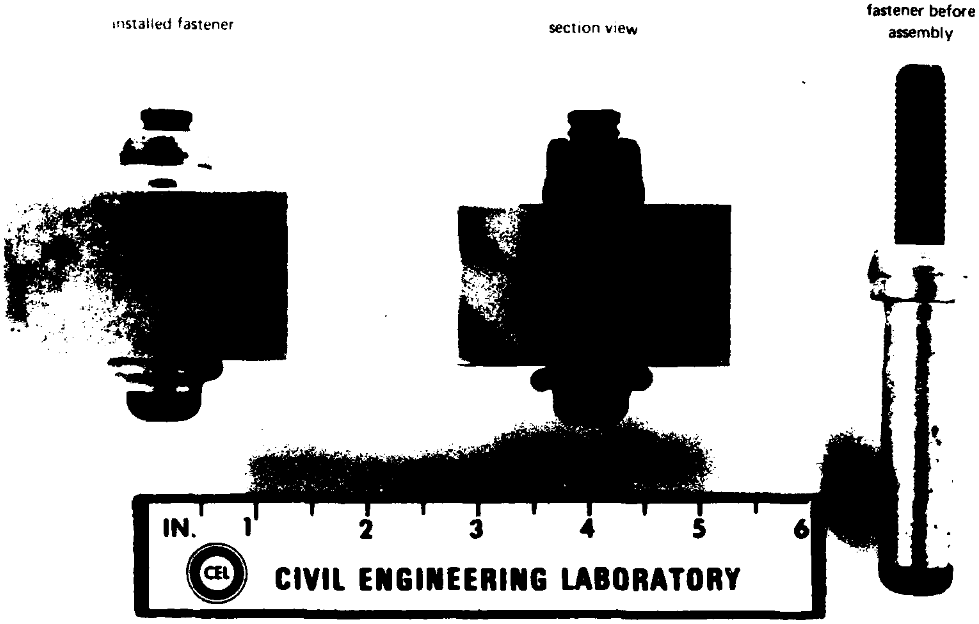

Figure 4-7a. Pull type blind hole fasteners.

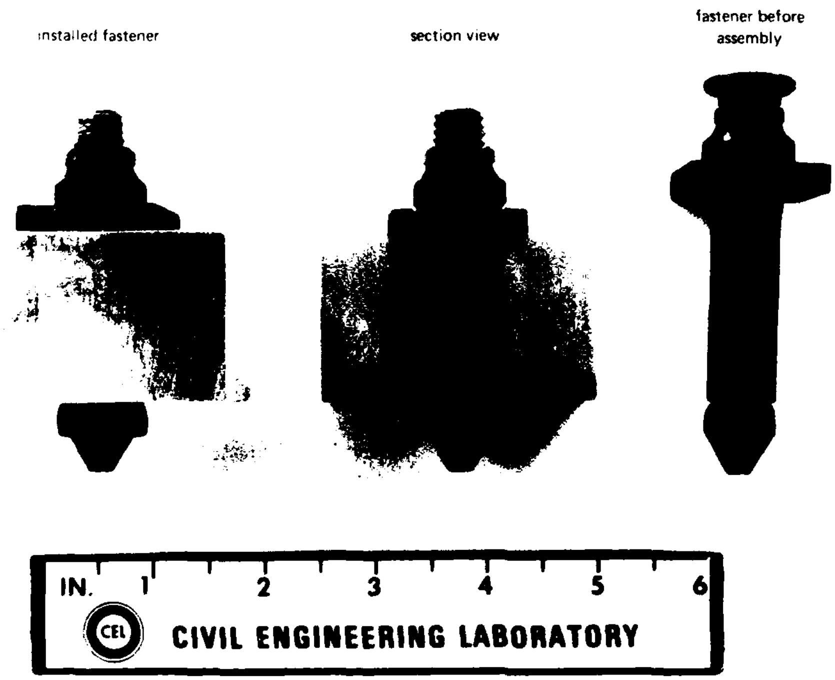

Figure 4-7b. Rotary type blind hole fasteners.

(3) should receive prime consideration by the design engineer; possibly, a combination of the two should be considered. For example, dragging split-piped cable back out to sea through a heavy surfzone and picking up at that point with an under-running vessel may keep the cable ship on-station a shorter period of time.

Generally speaking, the under-running vessel offers the best approach to applying long distances of split-pipe because, after the surfzone split-pipe is applied and hauled ashore, the cable ship can anchor the cable and start to sea. If less than 500 feet of split-pipe is to be applied, the drag-to-sea approach should receive prime consideration because, if resources are available, the pipe can be applied faster on the beach (expediting the cable ship's departure) and less at-sea operations are required.

The fourth technique, that of sliding lengths of split-pipe over the cable, is of questionable value because it can be replaced by one of the other techniques and it produces a few special problems (e.g., jamming up on the jute as the split-pipe is sliding out). Therefore, this technique will only be briefly discussed in this handbook to provide the reader with an understanding of the approach. More information on this technique can be found in the report Ocean Construction Experience Evolving from Project AFAR, published by NAVFAC.

Split-pipe installation typically starts above the maximum high-tide line on the beach and continues out to a depth where the hydrodynamic forces are not expected to be significant. An in-depth discussion on determining how much split-pipe should be applied can be found in Chapters 5 and 7 of this handbook. In some cases, isolated applications of split-pipe are used beyond the continuous *split-pipe run* (e.g., in critical areas like over cable splices and damaged sections and at suspension termination points).

Sometimes, it is advantageous to start continuous split-pipe run in two or more places and to work simultaneously (e.g., in a repair area where one must tie into existing split-pipe). When this is done, special split-pipe adaptor sections must be fabricated to connect the different runs; standard pipe half sections are cut to the desired length and welded together. The welding procedure for this work should not be considered trivial (described in Appendix A).

Similarly, to tie large-bore split-pipe into standard pipe, for going over a splice for example, special adaptor pipe sections must be made by cutting the appropriate pieces from supplied sections and welding them together as described in Appendix A.

If the subbottom profiles indicate that rock does not protrude above the cable route's lowest possible sand level, the split-pipe is only required across the beach and as far as vehicular or construction traffic may ever go. Even though the cable is piped in the beach area, it should be buried as deep as the water table or other constraints permit.

DIVER-APPLIED SPLIT-PIPE. The major advantage of the diver-applied split-pipe approach is its simplicity. Sophisticated equipment is not required, although it can be employed, for this type of operation. Also, the technique requires a minimal amount of coordination with other activities (e.g., cable ships). The technique is only recommended, however, when the application of split-pipe on the surface is not feasible.

1. Procedure - Best results are usually attained by staging split-pipe next to the cable prior to deploying divers. On sandy bottoms, only the amount of split-pipe to be used in a single day should be staged because scouring and sand transport can easily cover pipe sections. In some cases a whole pallet of split-pipe can disappear in 1 day. On rocky bottoms one can be more liberal with pipe staging; but if heavy weather, which can scatter the pipe, is forecast, appropriate precautions should be considered. If load-handling equipment is available, whole pallets of pipe can be deployed at appropriate intervals (approximately 60 feet). Diver-installed taught-line peanut buoys make an acceptable aiming aid. In shallow water, less than about 30 feet, the pallet can be dropped using a lanyard-released, pelican-hook assembly to speed the process. To ensure that the cable is not damaged by a falling pallet of split-pipe, the drop should be made approximately 10 feet to the side (perpendicular to the cable) of the marker buoy. Also, for cable safety, in water depths greater than 30 feet pallets should be lowered to the work area. Utilizing a slip line through the pallet's lifting eye will provide more control and will eliminate the need to put divers in the water to disconnect each pallet. Fasteners are best carried by divers to the work site along with the fastening tools.

Applying split-pipe in sand can be an exasperating experience. If pipe must be applied in the surfzone, optimum use of the tides should be made. If a "cherry picker," a crane, or a rough-terrain forklift is available, the cable can be gently lifted from the sand as far out as the equipment can reach.

CAUTION

ANY TIME SIDE LOADS ARE APPLIED TO A CABLE, CARE SHOULD BE TAKEN NOT TO EXCEED THE MINIMUM BENDING RADIUS OF THE CABLE (12 FEET IS FAIRLY SAFE). A BRIDLE ATTACHMENT OFTEN HELPS, ESPECIALLY IF ONE LEG OF THE BRIDLE CAN BE ATTACHED TO THE LAST SECTION OF APPLIED SPLIT-PIPE.

Some success has been attained by placing a cylindrical section (a cable reel about 1-2 feet in diameter, for example) under the cable. This raises the cable off the sand and reduces the amount of sand entrapped around the cable and between the half sections during assembly (Figure 4-8). A lift bag attached to the cylindrical section will facilitate movement along the seafloor by divers.

> CAUTION
>
> EACH ASSEMBLED SECTION OF PIPE SHOULD BE CAREFULLY CHECKED TO ENSURE THAT THE HALF-SECTIONS COME UP TIGHT AGAINST EACH OTHER.

If they do not, sand has been trapped inside (a very undesirable result because the holding capacity of the ball-in-socket connection is compromised if the sand should wash out). When this occurs the preload produced by the lockwasher will be lost, and the fasteners may vibrate loose (Figure 4-9).

In high surge areas, cable burial may occur very quickly. If split-pipe must be applied in this area, care should be taken to keep the cable on the surface of the seafloor. Float balloons can be applied to the cable to reduce its apparent density below 119 lb/ft^3 (1.9 g/cm^3).

Once the assembly process gets out of the heavy surge area, split-pipe half-sections can be successfully placed under the cable on a sand bottom by the diver's hands, which can wash a hole under the cable just ahead of the last applied section of pipe. However, the cylindrical lift section may still be desirable to assist in placing the bottom pipe half-sections. A long-handled pry bar will work satisfactorily to lift the cable for inserting the bottom section of pipe if adequate pry-bar footing can be provided.

The same technique is used occasionally to pry the cable, horizontally, away from outcroppings. When the cable is under tension and at suspension termination points, a lift bag, or several lift bags, will assist in repositioning the cable.

2. Support requirements - This technique for applying split-pipe requires little more than the appropriate fastener tools, some energetic divers, and the necessary diving equipment. Matters can be greatly facilitated if a LARC or similar amphibious craft fitted with a stiff leg crane is available.

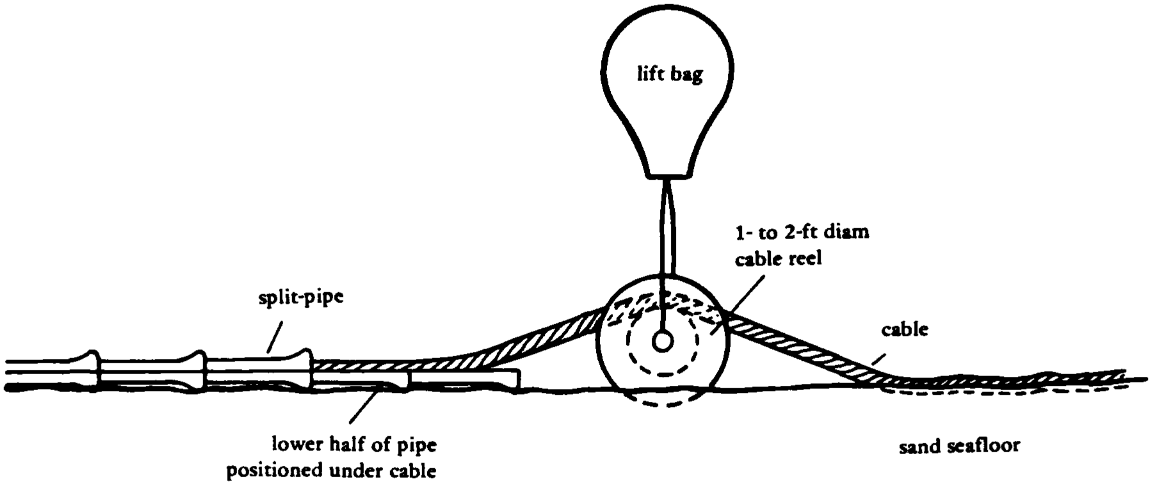

Figure 4-8. Cylındrical cable-lifting device.

Figure 4-9. Split-pipe failure due to loss of fasteners.

Manpower:* As few as six divers have successfully applied split-pipe to a cable when the protection extended only a few yards from shore in calm water, but the progress was painfully slow. The suggested number of working divers for a "typical" operation using this installation procedure is 16. Supervisory personnel and boat crew are also required, which will increase the number of on-site personnel. It is suggested that the boat operator either be a diver or be extremely familiar with working around divers. To a limited extent, increasing the number of working personnel will increase the rate of split-pipe application (assuming that support equipment for these people is available).

For this type of operation, no specialist or consultants are needed unless complex special fastener tools are employed.

Equipment: This type of operation does not require specialized equipment; however, the following major pieces of equipment are suggested:

Equipment	Requirements
Diving Equipment	as required
Appropriate Self-Propelled Floating Craft with Compass, Radio, and Fathometer	1 each
Diesel Hydraulic Power Source	1
Hydraulic Hose (100-ft length)	4
Hydraulic Impact Wrench	4
Hydraulic Grinder with Accessories	1
Cable Locator (MK 14 metal detector)	2 (if required)
Jeep with Sand Tires and Radio	1
Vehicle of Opportunity for On-Site Personnel Transportation	as required
Zodiac (14 ft with 25-hp outboard)	2
Recompression Chamber (if one not available on-site)	1
Transit with Accessories	2

*The information provided in this section is based on limited data from a few previous installations and as such is intended only as a guide in developing preliminary cost estimates. Since environmental conditions vary considerably from site to site the final decision on the number of personnel required to safely conduct the operation must be left to the discretion of the diving officer.

Equipment	Requirements
Builders Kit	1
Portable Tool Room	1
Split-Pipe (one-half section)	68 for each 100 ft required
Fasteners, Split Pipe	300 assemblies for each 100 ft required
Special Fastener Tools	if required
Radios	6 to 8

In addition to those items listed, the project planner may wish to review Appendix B-1 for suggested equipment and materials. Because every project is unique, Appendix B-1 is not intended to be either complete or exact but will aid in project planning.

3. Installation time estimates - Installation time will depend on the number of personnel available and diving conditions. With good to moderate conditions and proper staging of the split-pipe pallets, about 100 to 150 feet of pipe can be applied in an 8- to 10-hour work day. The use of blind bolt fasteners under the same conditions can almost double the application rate (100 to 120 ft in 5.5 hours).

4. Selection factors -

Bottom material and topography: Split-pipe is very difficult to apply on a sandy bottom in a high surge area (e.g., in surfzone where peak velocities are greater than 1-1/2 knot). If the cable is left unattended for even a couple of days in a high surge condition (sand bottom), it can bury in the sand to such an extent that removal will be extremely difficult.

Split-pipe is not necessary when the cable is lying on a sand bottom, and subbottom rock or coral will never be exposed.

Waves: To apply split-pipe in the surfzone, the surf must be less than 3 feet (preferably, less than 2 feet). Swells greater than 6 feet in water depth between 20 and 60 feet will pose application problems because of the heavy surge, which adversely affects diver performance.

Current: Split-pipe is very difficult to apply in currents or surges greater than 1 knot. This technique is not recommended if water velocities approach or exceed 1-1/2 knots.

Logistics support: An adequate diving support craft is needed for heavy extended underwater work. Normally, the previously listed diving support equipment can handle the distribution of split-pipe and the use of diver-operated power tools. Approximately 5,000 pounds of split-pipe and fasteners must be deployed for each 100 feet of cable to be protected.

Weather window: Weather conditions must be favorable enough to permit the diving team to get to the work area and stay on-station for long periods of time. For any significant amount of work to be done, a diving team must be able to stay on-station for at least 4 hours at a time. In good conditions (surge and current less than 1.0 knot, visibility greater than 3 feet, and depth less than 60 feet), approximately 8 to 10 diving station hours are required to apply each 100 feet of split-pipe.

Visibility: Reduced visibility (less than 3 feet) will reduce split-pipe application rates, especially if associated with surge or current conditions.

Hazards: Split-pipe augments the armor protection of a cable and improves the cable's ability to withstand marine organisms, anchor drags, trawler drags, and surge-induced abrasion.

Wind: Winds greater than 20 knots make diving operations difficult, especially if the wind is acting over a significant fetch to generate swell in the work area.

Design life: The design life of a split-piped cable system should not be affected by diver application.

Length of protected cable: Applying split-pipe underwater is a slow and arduous process. Long cable runs will require lengthy periods for application. In good conditions (surge/current less than 1 knot, visibility greater than 3 feet, and depth less than 60 feet) an average of 10 diving station hours are required to apply each 100 feet of split-pipe.

ONSHORE-APPLIED SPLIT-PIPE. Based on a study of forces required to pull cable and split-pipe along different surfaces, CHESNAVFACENGCOM (FPO-1) concluded that it is feasible to apply split-pipe to a cable on the beach and then have the cable ship pull the cable, with pipe applied, out to sea. To date, only one operational test of this procedure has been conducted, but it proved the approach was fundamentally sound. In general, the procedure calls for landing the shore end in a normal manner except that additional cable is pulled ashore. The length of additional cable is equal to the amount of split-pipe to be applied. Analysis shows that under ideal conditions approximately 500 feet of split-pipe can be dragged back to sea by a standard cable ship; in actual practice, only 330 feet has been so pulled. It is strongly recommended that further testing be conducted prior to future operations of this type are planned. Table 4-7 provides drag-resistance data generated for the feasibility analysis mentioned.

Table 4-7. Cable and Split-Pipe Drag Forces

Condition[a] No.	Condition[a] Description	Force (lb/linear ft)
Split-Pipe[b]		
1	On Dry Sand	50
2	On Wet Sand (submerged)	34
3	On Fiberglass Matting:	
	To initiate motion	30
	To continue motion	20
4	On Dry Sandstone	33
5	On Dry Granite	29
Cable[c]		
6	On Dry Sandstone	10.6
7	On Dry Granite	11.5

[a] Conditions 1 through 3 were obtained from SDC-1 inshore cable landing and stabilization – project execution plan FPO-1 report, 9 Jun 1972.

Conditions 4 through 7 were obtained from tests conducted at CEL in Apr 1975.

[b] 3-1/2-in. split-pipe was used for tests 1 through 5 with SDCL5 cable inside.

[c] SDCL5 cable with neoprene-coated armor was used for tests 6 and 7.

Hauling piped cable back out to sea is a good approach to handling the installation of split-pipe through a heavy surfzone. In fact, this approach is the recommended technique for handling situations where surf conditions are seldom less than 3 feet or when peak bottom surge velocities continually exceed 1.5 knots. Probably the major drawback to the technique is that the cable ship must stay on-station longer than normal. This is highly resisted by ship personnel, and precedent indicates that, without installation of a mooring system the cable ship (even with tug assistance) will, in all likelihood, not be able to maintain position in any more than quiescent conditions. The fact that the technique requires roughly twice as much time for the cable landing operation increases the probability that undesirable weather will be encountered before the cable is bottomed. If existing cables are in the vicinity, consideration should be given to how far the ship can safely drag its anchor, and contingency plans should be prearranged in case of this occurrence.

Another major consideration is beach configuration. Enough room is required to "inhaul" the additional cable and to apply the split-pipe on the cable. A beach topography that lends itself to this type of operation is also necessary. The slope of the beach should, and normally does, facilitate "outhaul" but should not be excessively steep for pipe application operations. Beach selection and preparation should eliminate berms or dunes from the hauling route; considerable outhaul resistance can be encountered if the cable is pulled through a dune or a berm. If the tidal zone is

rocky and irregular, the probability of split-pipe hanging up on the bottom during outhaul is significant until the water becomes deep enough for the floats to become effective.

1. Procedure - Standard procedures are used to beach the shore end. Then the cable must be pulled straight up the beach a length equal to the split-pipe that must be applied. If necessary, the cable can then be pulled around a sheave in an offline direction; this does complicate hauling procedures, however. The cable should then be secured to a deadman anchor directly in line with the incoming cable to prohibit any motion if at-sea loads are applied; any motion of the beach cable will considerably complicate pipe application operations. A quick-release device between the cable stopper and the deadman should be employed to facilitate release for outhaul.

Matting should be prestaged under the split-pipe application area to facilitate operations and to reduce friction between the cable and the beach (Figure 4-10). Care should be taken to prevent the cable system's snagging on matting edges in either direction of pull. Special precautions to prevent wear-through should be taken when it is necessary to have rises or humps in the hauling and application mat. The seaward termination of the mat should be snag-proof and wear-resistant. A significant amount of outhaul resistance can be encountered if the cable or piped cable wears through the mat.

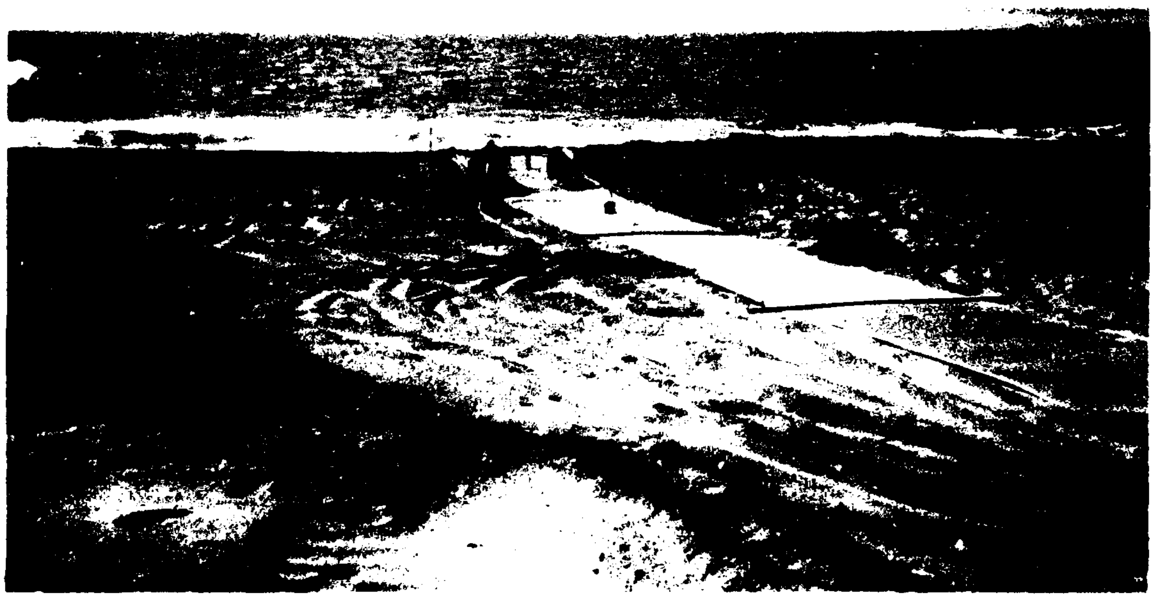

Figure 4-10. Beach matting for split-pipe application and haulout.

One of the major components of this technique is the beach guide. Beach guide designs used in standard cable landing operations are unsatisfactory. The beach guide should be able to handle the transition between unpiped cable to piped cable during the outhaul procedure. The guide should limit the cable radius to that specified as a minimum for the particular cable being landed. Figure 4-11 shows the configuration of the beach guide and matting. Until data are available on how much bending force split-pipe ball-in-socket connections can take, it is suggested that the

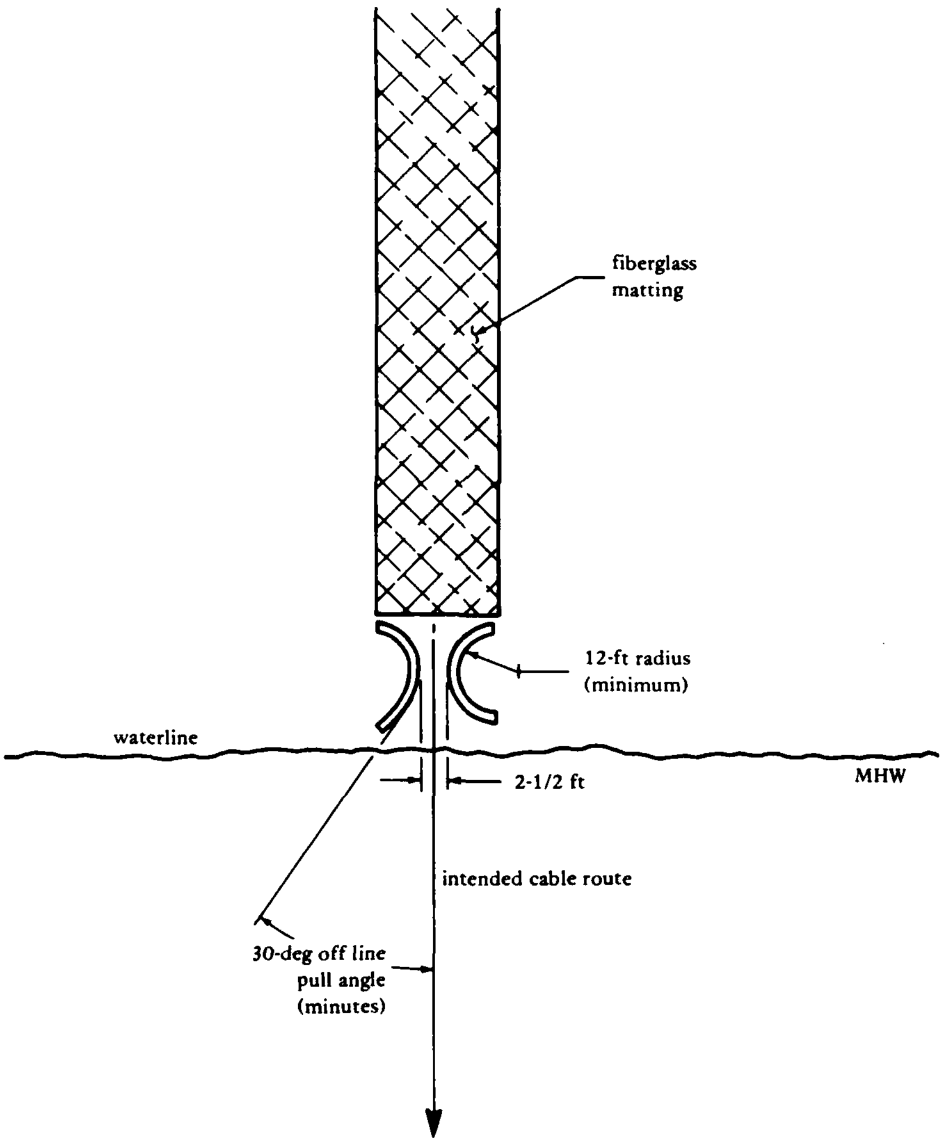

Figure 4-11. Beach configuration for typical shore applied split-pipe installations.

beach guide be designed for the split-pipe maximum turning radius of 12 feet. The beach guide should also be designed to facilitate the passage of split-pipe during outhaul even when offline pull angles result at the beach guide. The beach guide must be able to handle sideloads equal to the maximum that can be encountered during outhaul (i.e., when the cable ship is pulling at its maximum safe load for the cable and the offline pull angle at the beach guide is maximum).

Much detailed planning and prestaging of equipment should be made to minimize the split-pipe application time. This is essential to minimize the cable ship's time on-station and to reduce the possibility of bad weather interfering with operations. Adequate equipment, materials, and manpower should be supplied to minimize this time. Equipment and materials should be prestaged, with considerable thought going into their positioning. A dry run of the operation should be conducted so that every person knows the job perfectly. As with all ocean operations, contingency planning must be thorough.

During the planning and dry-run phases, innovative techniques should be developed and tested, if possible. Inhaul time can be saved if the grapnel rope is no longer than necessary. Optimum dozer positioning and utilization can save valuable time and improve preparedness for contingencies. A specially designed roller-topped dolly (Figure 4-12) can be used to run under the cable to facilitate insertion of the bottom split-pipe sections.

After the cable has been landed and secured to a deadman anchor, the split-pipe application can proceed. To ensure that the cable will not slide out through the split-pipe during outhaul, a shear gasket should be placed in the annular space between the cable and the split-pipe. Fire hose has been successfully used as a shear gasket in the past. It is suggested that only two bolts (those closest to the bell-end) be installed in each section of split-pipe until all or most of the split-pipe is on the cable. Then, all hands can undertake installing the remaining bolts. Also the split-pipe produces its connection strength from these two bolts; additional bolts are only redundant.

After the application of split-pipe, flotation must be applied to the piped cable for that portion in water deeper than about 3 feet. In addition to reducing the outhaul force as the cable begins to float, the use of flotation balloons assists in repositioning if the seaward portion of the floating cable is not in the desired location. If the standard 300-pound float balloons are used, at least 1 balloon should be applied for each split-pipe section (34 ballons/100 feet of piped cable). Leaders between the ballon and the cable should be kept short to float the cable in water as shallow as possible.

A dozer should be attached to the shore end of the cable to provide outhaul braking and control. A second dozer should be positioned to assist the cable ship in outhaul if problems develop (rigging should be ready to use).

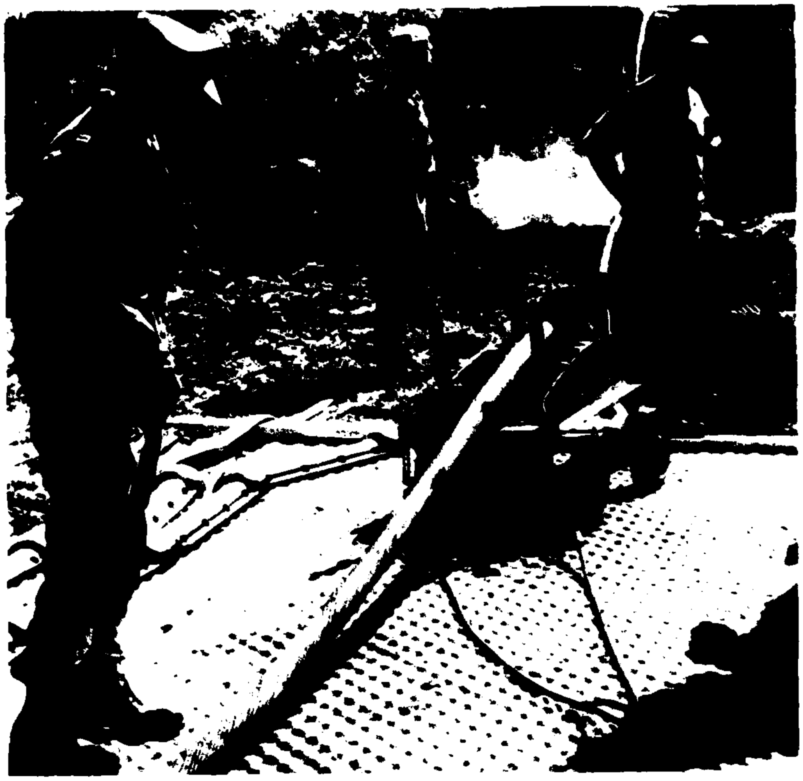

Figure 4-12a. Roller dolly and cable positioned at end of beach unit.

Figure 4-12b. Split-pipe being applied as cable comes off of back end of dolly.

Of course, during outhaul operation, good communications are extremely important between the ship, the beachmaster, and other key elements (like the dozers). The ship may want to have a secure communication link. An experienced damage-control crew should be on hand with cutting and burning equipment, pry bars, and miscellaneous rigging, to assist in clearing any rigging or pipe which becomes jammed in sheaves or the beach guide.

After the split-piped cable has been pulled out the desired distance, a stopper must be applied to the cable and secured to a deadman anchor on the beach. If the floating cable has developed an excessive catenary, at least 2 boats should be deployed during outhaul. Starting from the beach, the boats are used to pull the floating cable over the desired track (see Figure 4-13). As soon as the cable is over the desired track, swimmers cut the balloons, working seaward from the beach. As the swimmers approach the first boat, it should slip its holding line and move seaward of the next boat, this process continuing until the cable is safely bottomed. As the cable ship weighs anchor and starts laying cable to sea, a detail crew should begin collecting the float balloons while the remaining beach crew completes beach stabilization and burial. After the cable ship is a mile or so out to sea, the temporary cable stopper can be removed and the cable can be shackled into its permanent beach anchor.

2. Support requirements -

Manpower:* The underwater construction personnel on-site for a normal cable landing can accomplish this type of cable stabilization if they can augment the crew with 10 to 20 persons from a local command. The personnel list provided below can be used as a planning aid:

Personnel	Requirements
Beachmaster/OIC	1
Dive Supervisor	1
Radioman (Beach Control)	1
Dozer Operators	2

*The information provided in this section is based on limited data from one previous installation and as such is intended only as a guide in developing preliminary cost estimates. Since environmental conditions vary considerably from site to site the final decision on the number of personnel required to safely conduct the operation must be left to the discretion of the diving officer.

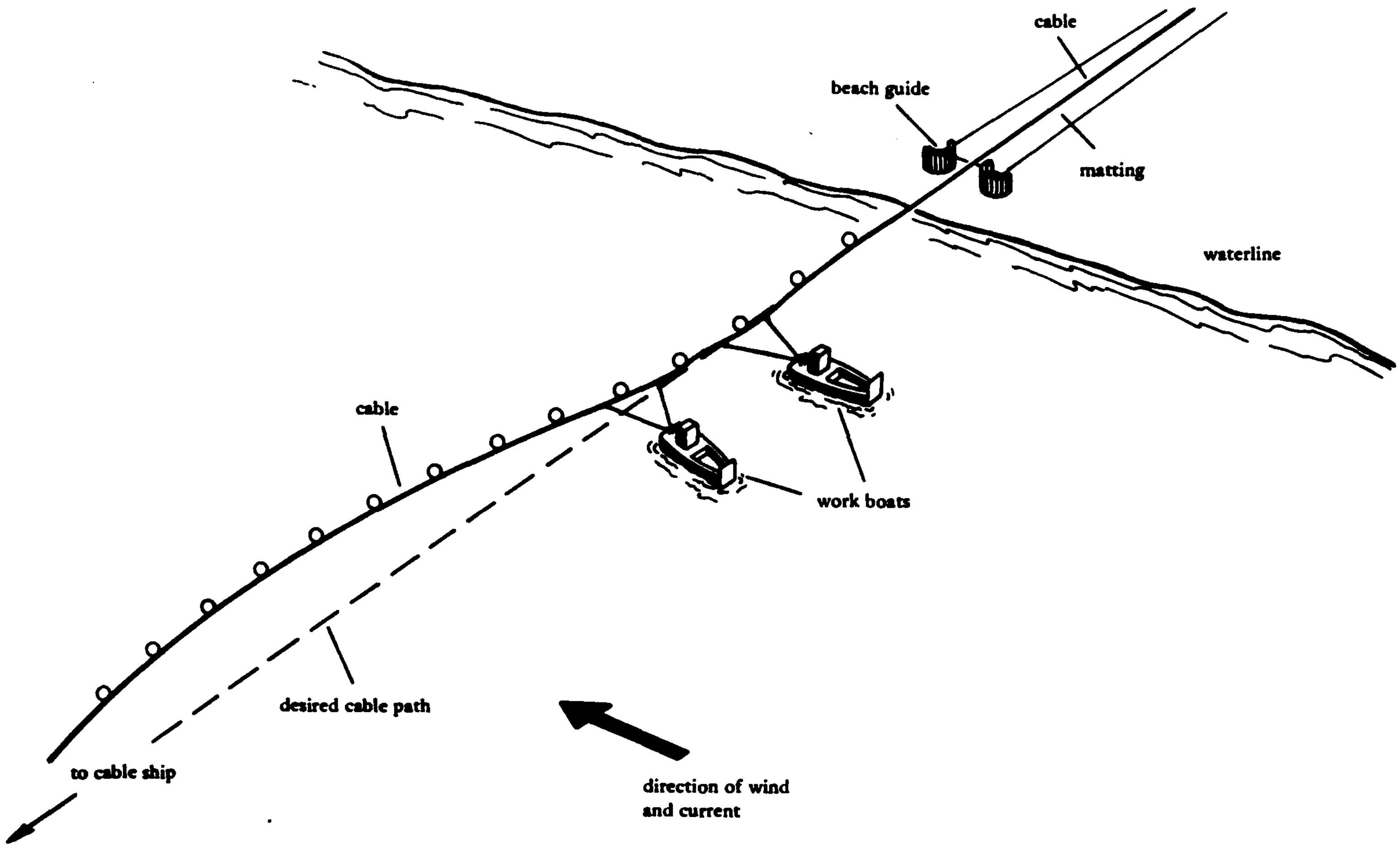

Figure 4-13. Removal of wind and current induced deflection of cable before positioning on seafloor.

Personnel	Requirements
Dozer Assistants	2
Damage Control/BTL Stopper Crew	2
Swimmers (Divers)	5
Split-Pipe Workers*	10-20
Boat Crew	as required for type and number of craft available

If floating craft are available, a diving-experienced boatswain's mate is valuable. All crew leaders should have previous cable landing experience, if possible. If the Officer-in-Charge (OIC) or the Assistant Officer-in-Charge (AOIC) is experienced in cable landing operations, no civilian consultants will be necessary unless special procedures are to be attempted or special complex tools are to be utilized.

Equipment: The required equipment for this type of operation will vary, depending on the site characteristics. A sample equipment list is provided in Appendix B-2. This list was developed from previous experience.

3. Installation time estimates - It is difficult, and probably unnecessary, to estimate the time required to prepare for split-pipe application only. Since this installation technique requires a cable landing, the time necessary to prepare for any standard cable landing (about 2 to 3 weeks) should not be significantly increased by adding the preparation requirements of this type of split-pipe installation.

If no major problems are encountered, a well-trained and disciplined beach crew can apply from 150 to 200 feet of split-pipe per hour. This rate can be increased with additional personnel or extremely good weather conditions.

All operations after outhaul are the same as those required for standard cable landing procedures and are not covered here.

4. Selection factors -

Bottom material and topography: This technique is not recommended for rock or coral bottoms unless a snag-proof chute is provided out to about a 3-foot water depth, where piped cable is floating.

No known data exists for drag forces of cable, grapnel, or split-pipe on clay.

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*Free hands when available work on split-pipe applications.

Waves: A surfzone with wave heights less than 3 feet is required to assure safe transit by LARC or divers.

Current: The shore-applied technique may be desirable in high fluctuating currents with a surge of 1.5 knots, where underwater application of split-pipe is extremely difficult.

Steady currents of 1 knot or more, that flow parallel to the beach, may pose severe station-keeping problems for a cable ship. This may necessitate prelanding installation of a multi-leg mooring system for use in keeping the ship in position while on-site. Extreme caution must be used in designing and installing such a mooring system to assure that it does not become entangled with the cable upon departure of the cable ship. Furthermore, these currents will produce pronounced catenaries in the floating cable and will most likely have to be straightened out before the cable can be sunk into place on the seafloor.

Logistics support: This is not significantly different from normal cable landing, other than providing the split-pipe, fasteners, and application tools.

Weather window: A weather window of 4 to 8 hours is adequate for this operation after proper beach preparaton and prestaging of equipment.

Visibility: Underwater visibility is not a factor in this approach, making this technique more attractive than underwater application in areas of low visibility.

Wind: Because of the effect of wind on the capability of the cable ship to stay on-station, this technique is not recommended when winds are forecast as >10 knots unless a mooring system has been preinstalled. This value could be increased to 15 knots if the wind is blowing directly along the cable track.

Length of protected cable: Applying more than about 500 feet of split-pipe in this manner would produce excessive outhaul forces on the cable. If additional split-pipe is required, multiple outhaul/split-pipe application evaluations will be necessary. The time for the cable ship to stay on-station would increase; and, depending on the time of year and latitude, the operating time may begin to exceed available daylight hours.

UNDER-RUNNING VESSEL. Several Navy immobilization projects have used the under-running technique. The technique has been used with the diver-applied technique, utilizing an LCM-6 during a cable repair and off of a barge during cable landings. Regardless of the size or sophistication of the operation the fundamental procedures and system components are the same.

This technique requires an adequate under-running vessel (an LCM-6 is about a minimum craft), and an under-running system must be installed on the vessel. The sea and weather conditions must favor a vessel being moored in the application area

for long periods of time. A nearby sheltered anchorage should be available, if possible. During cable landings, the time required to land the cable will have to be extended long enough to allow the under-running vessel to apply split-pipe to the surfzone portion of the cable. This prolongs the cable ship's on-station time slightly when compared to diver-applied pipe and increases the chances of weather-induced problems during cable landing.

The primary application of this technique is for installations requiring more pipe than can be accommodated using the shore-applied method.

The vessel must be fitted with a split-pipe application trough (with entry and exit chutes as shown in Figure 4-14), equipment and rigging to drag the cable across the deck (through the trough), a means to lock the cable in place in the trough, and a means to warp the vessel in a four-point moor. Also mooring legs will have to be implanted for the vessel.

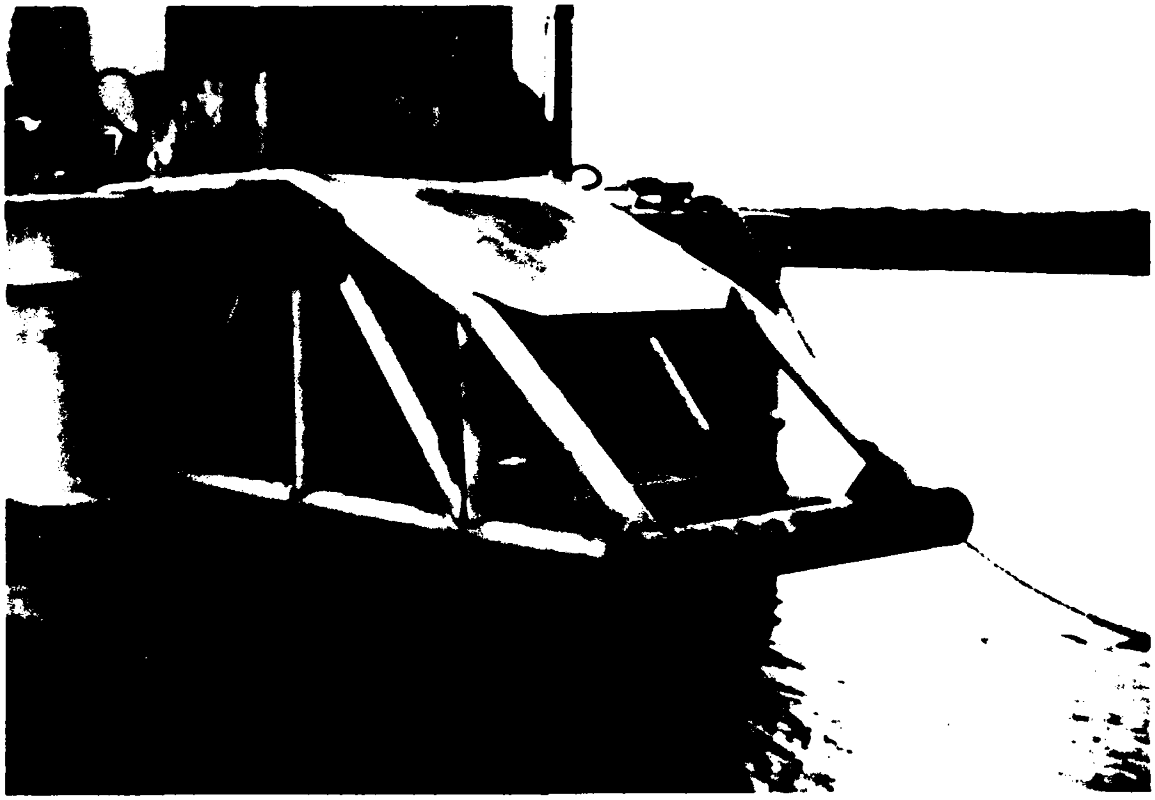


Figure 4-14. Split-pipe exit chute.

A 1/2-inch marine plywood deck should be adequate for most cable troughs. Heavy-gage sheet metal should be used over the plywood when a large amount of split-pipe is to be applied. Sheet metal can also be used without the plywood to smooth out irregularities on an existing deck. The entry chute should not exceed the cable's minimum bending radius and should deposit the cable approximately 2-1/2 inches above the trough deck. This 2-1/2-inch step will facilitate insertion of split-pipe half-sections. The step should be tapered, however, to permit warping piped cable back over and out the "entry" chute in case it's required. In some cases, a roller with retaining whiskers has been used instead of an entry chute. An air tugger or a hydraulic winch (1,000- to 2,000-pound capacity) should be installed for pulling the cable through the trough. Rigging should be provided to permit dragging the cable in either direction through the trough. As with shore-applied split-pipe, a shear gasket may be required in the annular space between the cable and pipe (depending on cable diameter) to prevent differential motion during pulling operations. A capstan head on the winch is often the best way to handle these operations. An additional winch with capstan head is required for warping the vessel around in its moor. A cable brake is needed to keep the cable firmly in place during pipe application. Two opposing breast lines attached to the cable and secured to cleats may serve the purpose (see Figure 4-15). A more sophisticated brake can be designed if desired, but it should be thoroughly tested before the operation.

Any structure required outboard of the cable trough should be removable to facilitate placing the cable over the side. In some cases the cable may need to be pulled up over the side into the trough; if this occurs, guide rails and hauling equipment should be provided.

1. Procedure - If split-pipe is to be applied during a cable repair, the repair vessel should be set up for under-running. After the last splice is made, the cable should be moved from the cable chocks to the trough. By use of the repair moorings, the vessel is warped as far as possible to one end of the repair section. The winch should be used to pull the cable through the trough. The movement can be assisted with vessel power.

After reaching the end of the repair section, split-pipe is applied to the cable lying in the trough. Then by dragging the piped cable through the trough, over the chute, and into the water, a new section of cable is exposed. During cable under-running, a control line opposing the winch drag line and slipped around a cleat may be used for control if needed. While applying the split-pipe the cable should be anchored in place in the trough with breast lines or a specially designed brake. This process is continued until as much of the repair section as possible is protected. The remaining split-pipe must be applied by divers. A special adapter section of split-pipe may be required to get the exact spacing for the tie-in to existing split-pipe. This adapter should be fabricated in accordance with Appendix A.

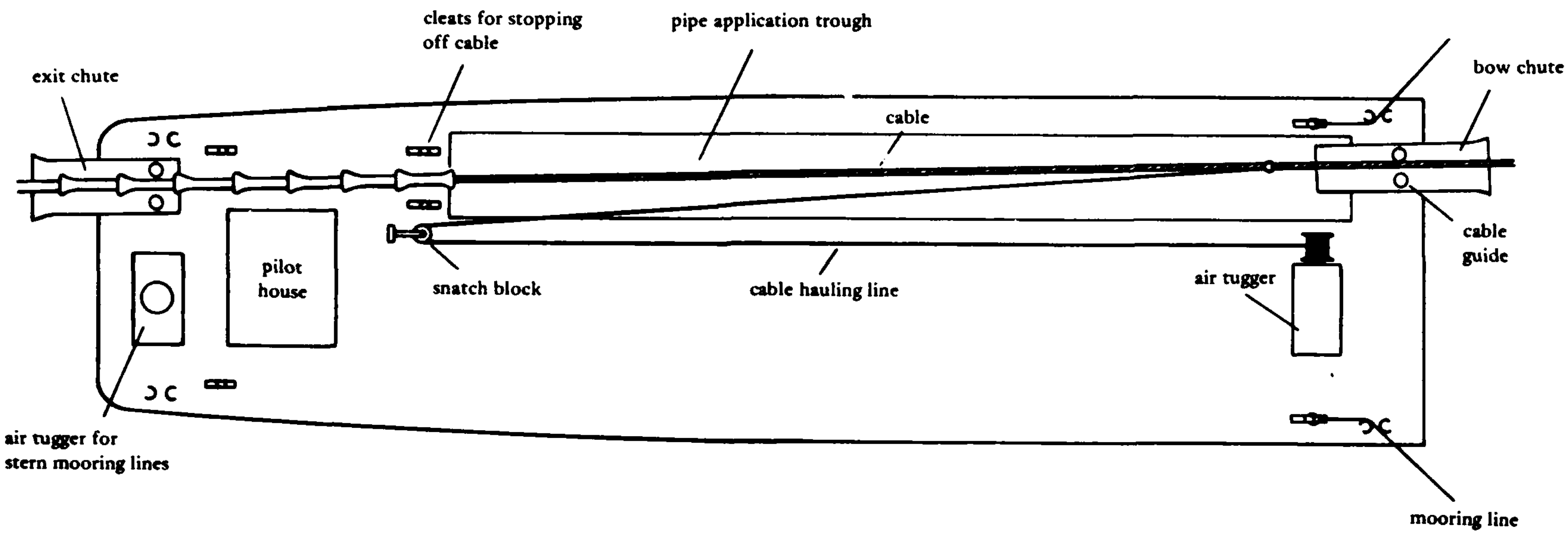


Figure 4-15. Typical LCM 6 configuration for under-running pipe application.

When the pipe is to be applied during a cable landing, preparations are made for a normal cable landing except for the following: (1) the split-pipe application vessel is moored just outside the surfzone on the cable route; (2) the grapnel rope running from the beach to the ship pickup buoy (grapnel buoy) may be passed over the split-pipe vessel's trough or rollers (Figure 4-16). During one cable landing/stabilization operation, a specially fitted YC barge (Figure 4-17) was used for both cable installation and split-pipe application. The designer should provide a positive technique to determine the proper point on the cable to start applying split-pipe. The cable ship should clearly mark this point on the cable; or, alternately, the point could be determined by the passage of the balloon cluster, at the beginning of the cable, past a marker buoy.

When the proper point reaches the application vessel, inhaul is stopped, and the cable is placed in the cable trough, if not already there. After the cable is secured in place to prevent movement, balloons are removed and split-pipe is applied to the cable. Flotation must then be attached to the split-pipe before it is hauled over the side toward the beach. If the standard 300-pound float balloon is used, one balloon per section of 3-1/2-in. ID split-pipe or one and one-half balloons per section of 5-inch split-pipe should be applied. The ballon crew and balloon riggers should understand the importance of their job. If three adjacent balloons should fail or if other comparable detrimental problems occur, then the cable can "domino" to the bottom with the obvious serious consequences. Leaders, between the balloon and split-pipe, should be kept to a minimum for that portion of the cable that will be in water less than 3 feet deep. The cable inhaul is then resumed to expose an unprotected section of cable in the trough. This process is repeated until the split-pipe reaches the beach.

After the above iteration process is used to apply split-pipe to the cable between the beach and the moored application vessel, the cable ship applies a BTL stopper to the cable and passes it over the bow rollers. To prevent the "domino effect," extra balloons should be applied to the cable shoreward of the BTL stopper at a distance approximately equal to twice the water depth. Roughly, one extra balloon is needed for every 25 feet of water depth. Divers or a boat crew then attach the BTL stopper to an anchor or a rock-bolt mooring. The cable ship is then free to weigh anchor and lay cable to sea.

Additional split-pipe is applied on the under-running vessel by warping seaward in its moor as the cable is pulled through the trough. Balloons are removed from the cable, and split-pipe is applied as discussed above until the cable is ready for bottoming.

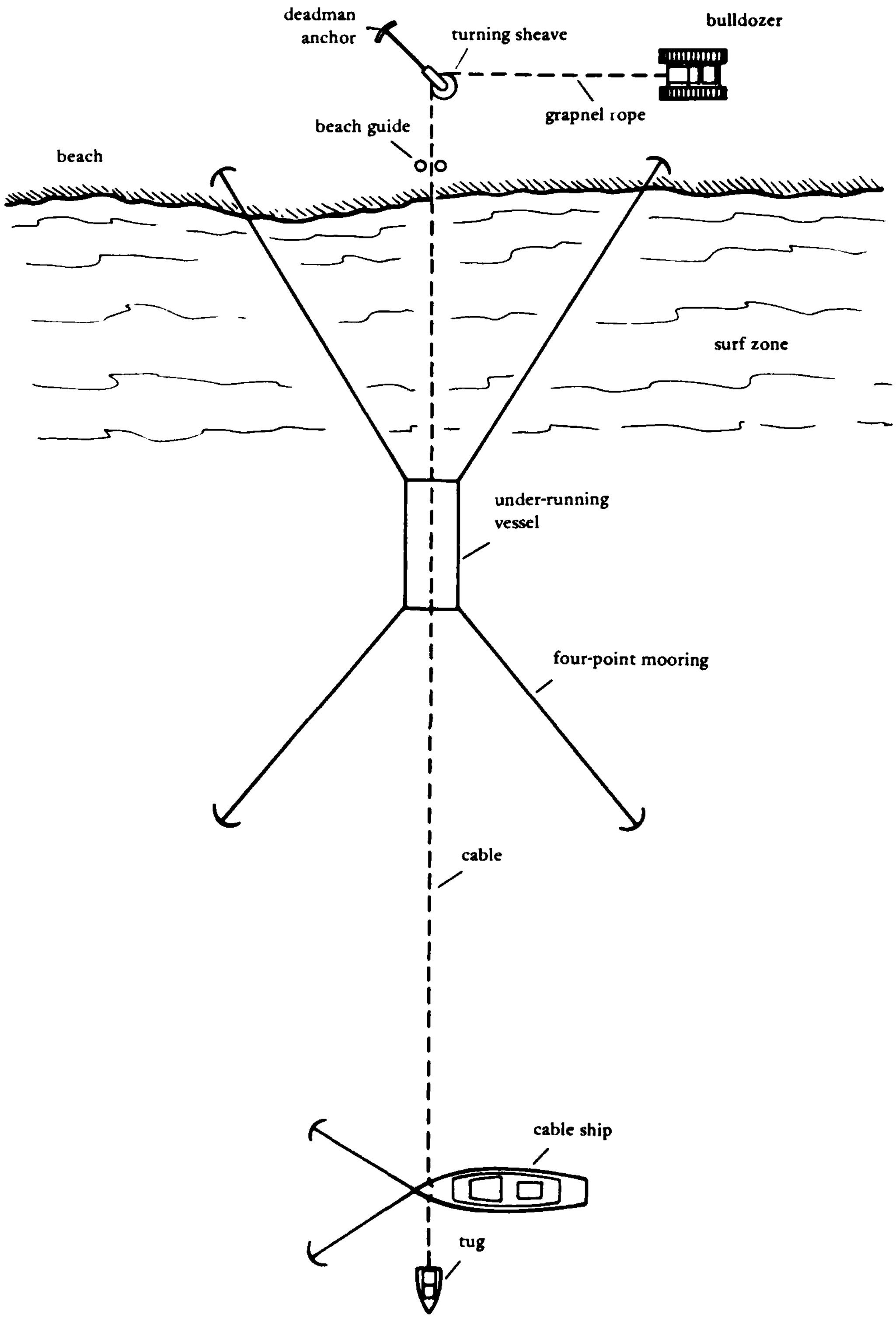


Figure 4-16. Typical equipment configuration for under-running vessel technique of applying split-pipe.

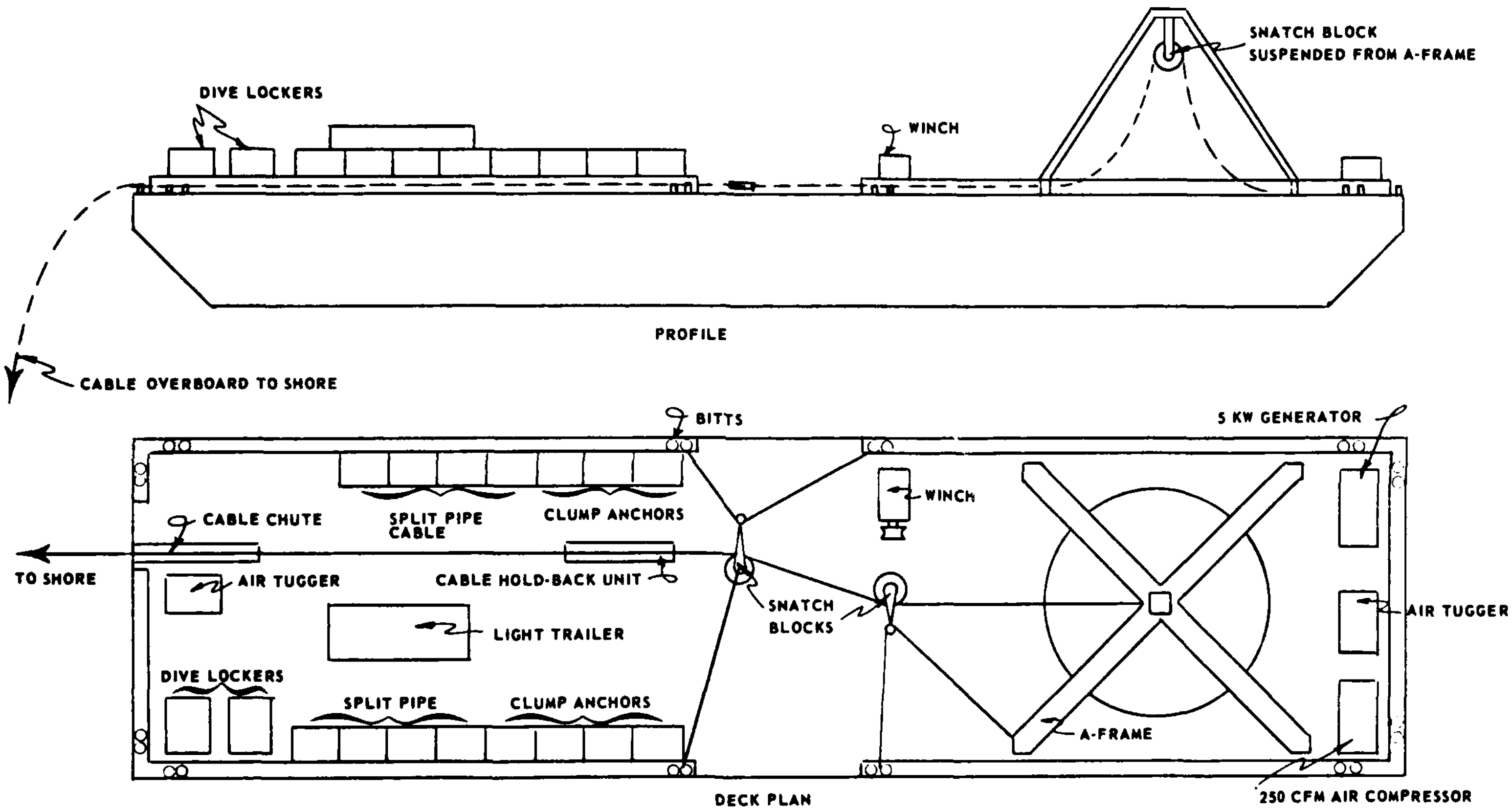


Figure 4-17. Barge deck layout.

2. Support requirements -

Manpower:* In addition to the personnel required for a standard cable landing, a crew is required for the under-running vessel. The crew should consist of about 14 men, 6 of whom should be experienced with underwater construction operations. If necessary, the additional 8 men can be carefully trained personnel from a local command.

Equipment: A specially outfitted, under-running vessel will be required: the longer the vessel, the more split-pipe that can be applied per mooring position - resulting in time saving. If the application vessel is unpowered then a "tug" will be required. A support/supply craft such as a LARC V is also necessary. Power tools and appropriate power sources for applying the split-pipe fasteners should be provided. Miscellaneous equipment and materials listed in Appendix B-1 and B-2 should be reviewed in the generation of a project list of material.

3. Installation time estimates - The rate that split-pipe can be applied in this manner will vary (depending on operating conditions, type of vessel used, etc.), but an average value of 75 ft/hr can be used for planning. Rates below this should be expected at first with the rate increasing to 100 ft/hr or more as experience is gained. These estimated rates do not include preparations and post application operations (e.g., bottoming the cable).

4. Selection factors -

Bottom material and topography: In irregular rocky or coral tidal zones, consideration should be given to the possibility of the split-pipe snagging during inhaul to the beach. If these conditions exist, techniques to unsnag the piped cable should be ready. The bathometry adjacent to the shore should not produce an exceptionally wide surfzone.

Waves: Wave and swell conditions should permit surf-zone transits by LARC and divers.

Current: This technique is more desirable than underwater application in areas where sea conditions produce bottom peak velocities >1-1/2 knots.

*The information provided in this section is based on limited data from a few previous installations and as such is intended only as a guide in developing preliminary cost estimates. Since environmental conditions vary considerably from site to site the final decision on the number of personnel required to safely conduct the operation must be left to the discretion of the diving officer.

As with the shore-applied technique, this approach will be adversely affected by strong currents (>1 knot). Strong currents will produce station-keeping problems for the cable ship, complicate mooring of the application vessel, and induce large horizontal catenaries in the cable.

Logistics support: The logistics of a standard cable landing would be complicated by the need for an under-running vessel. If the vessel is unpowered a "tug" will be needed to position the vessel in the mooring. A second craft would also be required for supporting and supplying the application vessel. Also, very substantial moorings must be installed for the application vessel if a sheltered anchorage is not immediately available.

Weather window: As with the shore-applied technique, the cable ship must stay on station longer, and sea and weather conditions must favor mooring the application vessel just seaward of the surfzone. On the other hand, the overall project can be completed much faster than underwater application.

Visibility: Since the split-pipe application is done on the surface, underwater visibility is not a factor for this technique. This technique may be more desirable than underwater application in areas with poor underwater visibility (<3 feet).

Hazards: The hazards discussion in the Diver Applied section applies.

Wind: Wind will adversely affect the capability of the cable ship and the application vessel to stay on-station. The discussion in the Shore-Applied Split-Pipe section applies.

Length of protected cable: This technique's attractiveness improves as the length of cable requiring protection increases.

DRAG-OVER-CABLE. As discussed in the comparison of available techniques, this approach is of questionable value. However, since the technique has been successfully used, it will be briefly discussed here in case of some future application. For more information on the technique the reader is referred to the CHESNAVFACENGCOM (FPO-1) report Ocean Construction Experience Devolving from Project AFAR (NAVFAC, no date).

The technique calls for assembling approximately 60 feet of split-pipe at a time over the cable on the beach. Each 60-foot length is then dragged over the bottomed cable to its desired resting place. Bottom material and topography determine whether or not a sled may be required to facilitate dragging the pipe to sea. A special transition pipe section (standard section with the connecting bell section removed) is applied to the shoreward end of each length of split-pipe. After the length of split-pipe is pulled up solid against the preceding assembly, the transition section is removed and replaced with a standard section by divers.

The process can be quite slow due to spalling of the cable's jute covering. Divers can be used to monitor the dragging progress and to cut away spalled jute. Approximately 400 feet of split-pipe can be applied per week in this manner.

4.2.3 Oil Field Pipe

Background. The primary protective function of oilfield pipe is to provide rigidity to underwater cables and thus mitigate cable abrasion produced by oscillatory water motion. For this reason the technique may find an application in protecting single or multiple runs of small cable.

In most cases the use of oil field pipe will actually decrease the cable system density or increase the ratio of surface area to weight, making the system more sensitive to hydrodynamic forces. Therefore, adequate pipe immobilization (see Section 4.3 and Chapter 6) is necessary in all but quiescent conditions. Underwater suspensions should be kept less than 30 pipe diameters if possible. This means that the bathometry should be relatively regular.

Oil field pipe is a tough pipe that resembles standard pipe for all practical purposes. Lengths approximately 20-feet long are convenient to work with on cable protection projects. A threaded coupling is used between pipe sections. To facilitate assembly and attain the desired joint strength and torsional resistance, a pipe assembly machine is used to assemble the desired length of protective pipe. The cable to be protected is run inside the pipe during or after assembly of each length. The internal portion of the pipe can be filled with something other than seawater. A protective and insulatory medium will provide additional corrosion protection, or cement grout can be pumped in to increase the in-water weight.

Procedure. A pipe assembly machine, used by the Amphibious (PHIB) Seabees for fuel and water line installations, works well to assemble oil field pipe for cable protection. The standard PHIB Seabee practice is to start at the beach, assembling pipe and laying it on the bottom as the assembly is pulled to sea. This procedure should work as well for cable protection, but beach assembly was used in the one cable protection test made by the Navy. The beach assembly approach will be discussed in this section.

The beach should be graded to provide a smooth area as long as the assembled pipe is going to be and is in line with the cable route. The pipe assembly machine and pipe sections are placed at the shoreward end of this area. A sled carrying the cable reel is placed behind a bulldozer and secured to the seaward end of the pipe assembly. Figure 4-18 illustrates a layout of a possible pipe assembly area. Before each section of pipe is screwed onto the assembly, a feeder shaft is used to pull the cable through the pipe section. The cable is then secured to a deadman through a "come-along" before the dozer advances the pipe assembly so the next pipe section can be added.

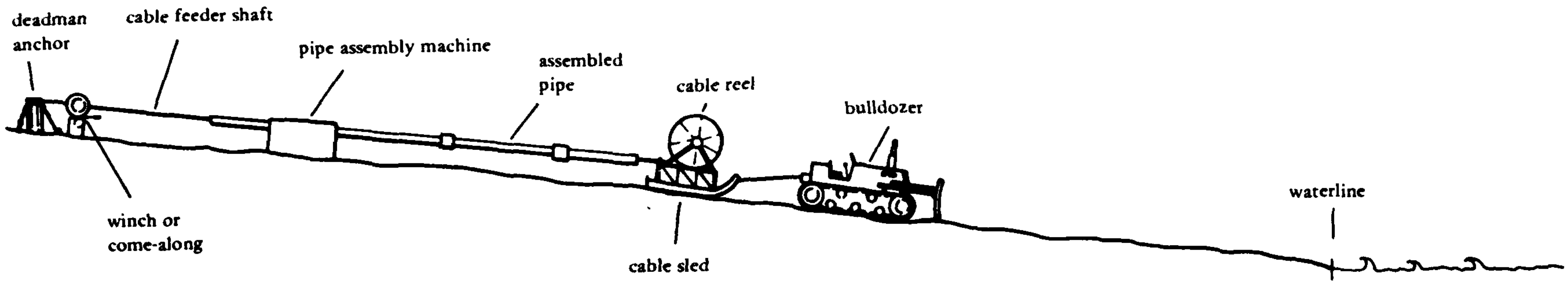


Figure 4-18. Pipe assembly area.

A come-along or winch is needed to take up losses incurred during each disconnection or connection. Good communications are needed between the dozer operator and the pipe machine operator. As the dozer advances the pipe assembly, the cable reel feeds the cable into the seaward end of the pipe. This process is repeated until the desired length of pipe is assembled on the beach.

After the pipe is assembled, with the cable inside, the ends should be sealed to provide extra buoyancy during the outhaul, and appropriate flotation should be attached to the pipe with short attachment leaders. Outhaul assistance should be available on the beach to assist if the outhaul vessel is unable to handle the load. A LARC can be safely driven over the pipe to straddle it. The pipe can then be belly strapped to the LARC for pulling assistance.

If significant wind or current may be encountered (parallel to the beach), then a means for providing lateral support of the floating pipe should be considered. A good sized boat may provide this restraining force. Although oil field pipe is quite strong and surprisingly flexible, a severe floating catenary may kink the pipe and fault the cable.

A floating craft with an adequate winch should be moored just seaward of the pipe termination point. Care should be taken in the design and installation of the mooring system. The moorings should be able to withstand the required force to pull the pipe off the beach, besides holding the craft in strong on-shore winds.

By use of the floating craft's winch, the pipe with internal cable is pulled off the beach and out to sea to the desired point. Starting from the beach, swimmers then cut balloons from the pipe to bottom it. The pipe can then be flooded with seawater or other desired medium, making the system ready for pipe immobilization and seaward cable hookup. Normally the seaward end of the pipe would be attached to some type of anchor.

To give the designer a feel for the drawbar pulls required, approximately 9,000 pounds of force was required to pull about 1,400 feet of 4-inch pipe down a 2-degree beach slope.

Support Requirements.

MANPOWER.* This procedure has been done with approximately 11 men. If construction personnel are not familiar with the pipe tongs, either special training or a consultant will be needed.

*The information provided in this section is based on limited data from a few previous installations and as such is intended only as a guide in developing preliminary cost estimates. Since environmental conditions vary considerably from site to site the final decision on the number of personnel required to safely conduct the operation must be left to the discretion of the diving officer.

EQUIPMENT. This procedure requires a pipe assembly machine, a dozer, a cable reel sled, a vehicle to get the pipe to the beach, a LARC, a small boat with 100-hp (or greater) engine, a zodiac with 25-hp outboard, a large winch capable of producing the pulling force needed and capable of holding the length of adequate wire, and a craft to support the winch. A crane or stiff leg may be necessary to emplace the pipe termination anchor or any instrument package required. The following is a list of equipment and materials used during the previous installation.

(1) Surface craft (LCU or similar)

(2) Foster Cathead air tongs

(3) Truck crane for surface craft

(4) Winch for surface craft

(5) 3,000-ft 1-1/4-in. polypropylene line, 19,000-lb breaking point

(6) 3,000-ft 1-in. nylon line, 22,000-lb breaking point

(7) 3 moorings for LCU, 910-lb Navy stockless prime anchor

(8) Threading pipe for cable pulling 25 ft

(9) End coupling for towing the pipe

(10) Stuffing gland for final sealing

(11) Bracket to hold pipe to concrete anchor

(12) LARC

(13) Radio communications

(14) Reel for rewinding the electrical cable

(15) Reel and sled

(16) Load cell system for towing

(17) Shore-based junction for pipe end, concrete

(18) Diver-to-boat communications

(19) Pipe (20- and 24-ft sections)
- 4-in. deflection in 30 ft without permanent set
- Tensile strength, 43,000 psi
- Seating torque, 2.5-3K-ft-lb
- Radius of curvature, 2500 ft
- Rockwell A scale, 52

(20) Tractor

(21) 300-lb floats

Installation Time Estimates. The time required to grade the beach for pipe assembly operations depends on beach topography. Approximately 1,400 feet of 4-inch oil field pipe can be assembled on the beach in 1 day. Another day is needed to set the outhaul craft's moorings and prepare the pipe for outhaul. About 1 day is needed to outhaul the pipe, sink it to the bottom, and install the termination anchor with instrument package if appropriate.

Selection Factors.

BOTTOM MATERIAL AND TOPOGRAPHY. This technique can be utilized for any bottom material. If the bottom is not smooth sand then further immobilization should be provided. Problems in sand may also occur if the system density is less than 119 lb/ft^3 (1.9 g/cm^3); self-burial will not occur. Oil field pipe is unable to conform to a highly irregular bathometry, and suspensions greater than 30 pipe diameters should be avoided if possible. The beach topography should enable preparation of the pipe assembly area.

WAVES. Sea conditions should permit surfzone transits in the LARC and a YFU landing if the site cannot be logistically supported from shore. A surf of 3 feet or less was found acceptable. Also, sea conditions must favor mooring the outhaul craft in the desired location; surf in this area would not be acceptable, and swells should be less than 6 feet.

CURRENT. Strong lateral currents (>1 knot) will produce mooring problems and complicate catenaries in the floating pipe during outhaul. If strong currents with a peak surge or steady force >1.5 knots are present at the seaward pipe termination point, then techniques should be designed to enable surface installation of the pipe termination anchor and instrument package (if required).

LOGISTICS. Preferably, the beach should be accessible from inland for delivery of the heavy equipment (e.g., dozer, pipe tongs) and material. If not, conditions should permit landing this material from the sea.

WEATHER WINDOW. Approximately 10 hours of good weather is needed to outhaul 1,400 feet of pipe and place it on the bottom.

VISIBILITY. Except for instrumentation hookup and pipe termination anchoring, underwater visibility is not a significant factor.

HAZARDS. Oil field pipe considerably enhances the ability of small cables to withstand marine organism attack, anchor drag, trawler gear snags, and abrasion. Scour burial in the bottom sediment is desirable to remove the system from hydrodynamic forces.

WINDS. Winds <20 knots would be desirable for beach operations and placement of outhaul craft moorings. Wind should be <10 knots during outhaul operations.

LENGTH OF PROTECTED CABLE. Continuous lengths of oil field pipe >1,500 feet will demand greater availability of equipment and beach assembly area.

4.2.4 Concrete

Background. A variety of techniques have been employed in the placement of concrete on the seafloor for stabilizing cables. These techniques can generally be grouped into three categories: (1) sacked concrete, (2) cast-in-place concrete, and (3) precast elements.

Sacked concrete consists of numerous flexible containers such as burlap, nylon, canvas, or rubber bags filled with concrete on the surface and positioned over the cable on the seafloor. Cast-in-place concrete consists of large masses of concrete delivered to the seafloor as a wet mix from the surface and either allowed to flow as an unconfined mass over the cable and seafloor or pumped into flexible forms (bags) which have been prepositioned over the cable. Precast elements are monolithic blocks of reinforced concrete which have been poured into forms and allowed to cure on land before positioning over the cable on the seafloor. The latter technique has been used primarily for stabilization of positively buoyant submarine pipelines in relatively calm water. Precast concrete elements, however, have been used in conjunction with other mass anchor techniques (such as chain, Section 4.2.5) for the stabilization of cables where the concrete does not come in direct contact with the cable.

Although a great deal of literature exists on methods of placing concrete under water, virtually nothing could be found which documented the results of actual cable stabilization installations or how successful concrete was at immobilizing the cable for any extended period of time.

Description. Concrete is a composite material which consists of a binding medium within which are embedded particles or fragments of a relatively inert mineral filler. In portland cement concrete, the binding material is a combination of portland cement and water, commonly called cement paste. The filler material is aggregate and consists of particles of stone ranging in size from fine sand to gravel several inches in diameter (Troxell et al., 1968).

Five types of portland cement are currently in use in the United States (Table 4-8). Type II is generally preferred to underwater applications because of its resistance to deterioration from sulfate ions in seawater (Myers et al., 1969; Svanberg and Cox, 1973). The water-cement ratio and size and amount of aggregate will vary, depending on the technique used to deliver the concrete to the seafloor. The ratio of water-to-cement affects both the workability and the final strength of

Table 4-8. Current Types of Portland Cement

General Characteristics	Designation	Use
General-purpose or normal	Type I	For general concrete construction where special properties are not required
Modified general-purpose	Type II	For general concrete construction where it is exposed to moderate sulfate action or where the heat of hydration must be somewhat lower than for normal cement
High early-strength	Type III	For use when rapid hardening is required
Low-heat	Type IV	For use where the heat of hydration must be a practicable minimum
Sulfate-resisting	Type V	For use where high resistance to the action of sulfates is required

the concrete. Since the use of concrete for stabilizing cables does not require a high strength mix, workability usually governs the water-cement ratio. The "slump" of the concrete is a pratical method of approximating the consistency or workability of concrete mixes (Troxell et al., 1968). Figure 4-19 gives the approximate free-mixing water requirements for different slumps and maximum sizes of aggregate.

Conventional concrete has an in-water weight of between 80 and 100 lb/ft^3. Heavyweight concrete has been used in some underwater construction applications where additional deadweight load is required. This additional weight is obtained by using iron ore (magnetite) for the aggregate. This produces concrete with an in-water weight over 160 lb/ft^3. The added cost of shipping heavyweight aggregate to the site, rather than using local material, may not be economically feasible.

Admixtures are substances used in concrete for altering its normal properties. Most uses of admixtures are typically to (1) improve workability of fresh concrete, (2) alter the setting time, (3) reduce the amount of water required, and (4) to improve durability by entrainment of air. The most commonly used admixture for underwater application is an accelerator such as calcium chloride or sodium silicate, which decreases the initial setting time and thus minimizes the possibility of hydrodynamic disturbances to the concrete. Care must be taken when using accelerators that the time required to deliver the concrete to the seafloor is not longer than the set time. Sacked cement can be purchased with accelerator already added. If an accelerated set is desirable the use of cement with premixed accelerators is advisable to assure uniformity of the setting times.

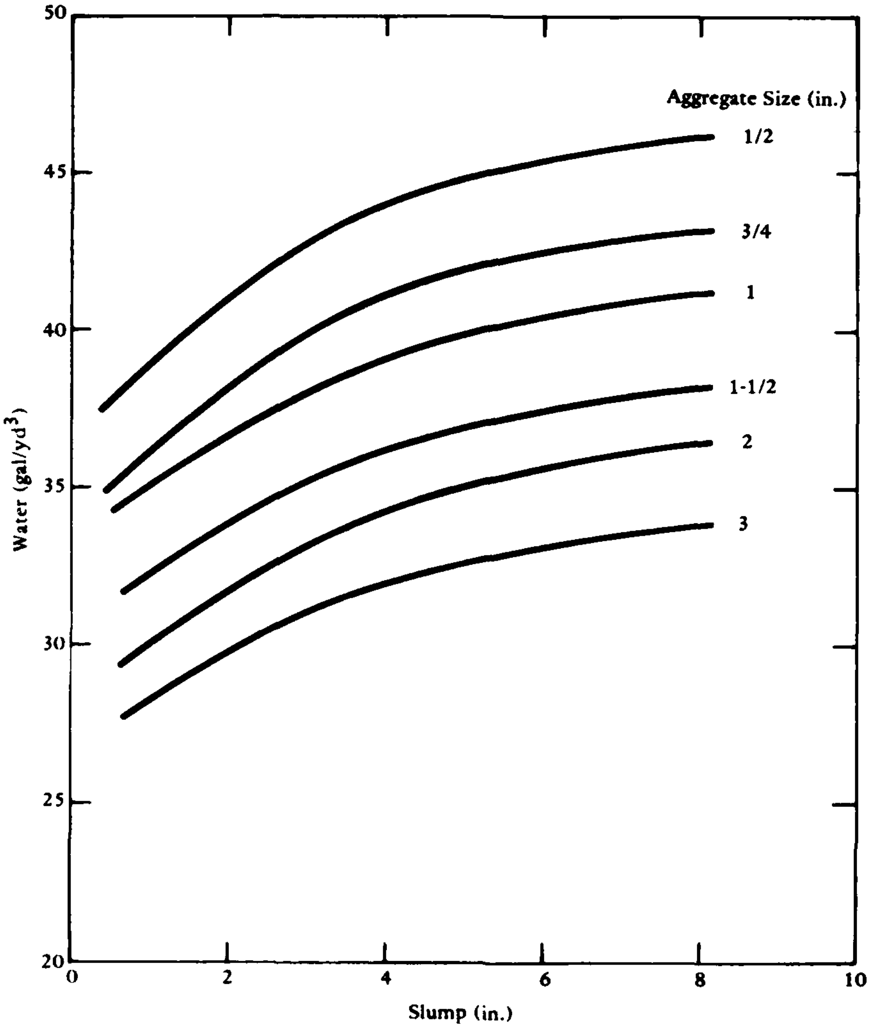


Figure 4-19. Approximate requirements of free mixing water for different slumps and maximum sizes of aggregate (derived from: Composition and Properties of Concrete, by G. Troxell, et al. McGraw-Hill, 1968, New York).

SACKED CONCRETE.

1. Procedure - Two basic techniques have been used for this type of stabilization. In the first, burlap bags are filled about half full with dry-mix concrete, and the ends are securely fastened. The bags are lowered to the seafloor and positioned over the cable by divers. In most cases, several bags of concrete will be positioned in one spot to provide the necessary weight to counteract the hydrodynamic forces. Pieces of rebar should be driven through adjoining bags (Figure 4-20) to assure that they remain together and act as a single mass. The concrete is hydrated as seawater soaks through the burlap and is absorbed by

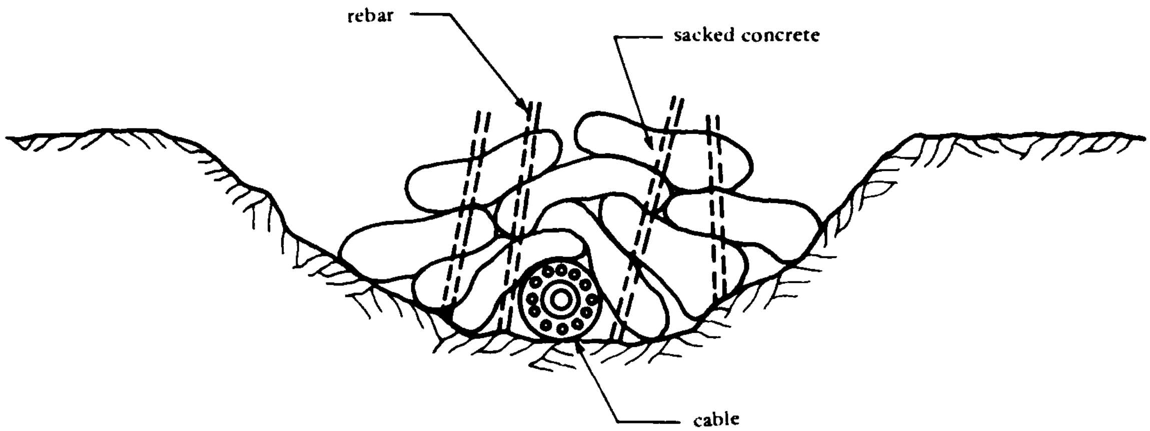


Figure 4-20. Configuration of typical sacked concrete stabilization system installation.

the concrete. An advantage to this method is that no large mixing machinery is required on-site, and the setting time is quite slow because of the slow hydration process, which allows plenty of time for handling and placing the bags. The burlap bags also prevent to some extent the concrete's being washed away by current and surge action.

The second method uses a low-slump wet-mix of concrete placed in burlap or jute bags on the surface support craft. The bags are filled about two-thirds full, and the ends are securely tied. These are then lowered to the seafloor and positioned by divers. The handling time for this technique must be shorter than for the first technique because the concrete has been premixed. However, full hydration is assured, a good bond between bags can be achieved, and the general quality of the mix can be verified (Myers et al., 1968). This technique is particularly useful if a short weather window is anticipated; with the dry-sack method the concrete may not be fully hydrated and could be displaced by wave action.

2. Support requirements -

Manpower:* Personnel requirements will depend on the extent of the stabilization operation, water depth, and the length of time allowed for completion. As a minimum the following personnel should be on-site:

*The information provided in this section is based on limited data from a few previous installations and as such is intended only as a guide in developing preliminary cost estimates. Since environmental conditions vary considerably from site to site the final decision on the number of personnel required to safely conduct the operation must be left to the discretion of the diving officer.

Personnel	Requirements
Divers	5
Diving Officer	1
Boat Operator	1
Load Handlers (concrete)	4

Equipment: This technique required a minimum of specialized equipment; the major item is a surface support platform capable of handling the weight and bulk of the concrete required for at least 1 day's operation and equipped with load-handling equipment that can lower 8 to 10 bags of concrete at a time. The following lists the items needed to support this type of operation.

	Equipment	Requirements
1.	Surface Support Platform LCM-6, LCM-8, barge, etc.)	1
2.	Hoist/Crane (1 ton, minimum)	1
3.	Concrete	as required
4.	Burlap Bags	as required
5.	Rebar	as required
6.	Diving Gear	as required
7.	Concrete Mixer	1
8.	Shovel	4
9.	Water Storage Tank (500 gal, minimum)	1
10.	Submersible Pump and Hose	1
11.	Electric Generator	1

Items 7 through 11 are only required if wet-mix concrete is to be used.

3. Installation time estimates - No data were available on the installation times needed to apply sacked concrete during actual cable stabilization operations. The total time, of course, depends on the number of sacks required at each spot along the cable and the spacing between stabilization points. The use of wet-mix

concrete also requires additional time because the concrete must be mixed just prior to placement and cannot be stockpiled in advance as dry-sacked concrete can.

4. Selection factors -

Bottom material and topography: The use of sacked concrete is feasible only on rock or coral seafloors where self-burial of the sacks or scouring of the sand under the sacks is not possible. Since no actual mechanical bond exists between the sacks of concrete and the cable, the differential density between the cable and concrete would almost certainly cause them to separate on sand or clay seafloors. Topography has very little impact on the feasibility of this technique.

Waves: Waves can cause problems both during the installation and during the operational life of the system. During installation, excessive wave action can cause the cement to leach out of the concrete, creating laitance and segregation and resulting in very poor bonding between the bags. In the extreme, the bags may be displaced from the intended immobilization spot before the concrete can set. Wave-induced surge >1 knot will make it difficult for the divers trying to position the sacked concrete on the cable. During the life of the installation, large storm waves (on the order of 20 to 30 feet high) may cause the sacked concrete to be unstable and slide along the seafloor (Valent et al., 1975a).

Current: Excessive current has the same effect on this technique as waves.

Logistics: If the concrete cannot be obtained locally, the weight and bulk of the concrete may create extensive shipping problems. Once on-site, the dry cement must be stored in an area where it is not exposed to moisture and rain.

Weather window: The weather window must be sufficiently long to allow placement of all the bags on the seafloor and allow the concrete to set before being exposed to hydrodynamic forces. Ideally, this should be at least 1 to 2 weeks after the last of the concrete has been positioned on the seafloor.

Visibility: Visibility has very little impact on this technique since the cement which leaches out of the bags will tend to reduce local visibility. Initial visibility of <3 feet will slow the diving operation and may present a problem for the divers in finding the cable and positioning the concrete before it begins to set.

Hazards: Sacked concrete provides little or no protection against any of the hazards except hydrodynamic forces and then only if maximum wave conditions are expected to be <20 feet during the life of the installation (see preceding discussion on waves).

Wind: Winds greater than 20 knots make diving operations difficult, especially if the wind is acting over a significant fetch.

Design life: Because it provides little protection against hazards and because its stability is questionable in the presence of large waves, sacked concrete is not recommended to stabilize cables that are part of a critical installation or have a design life >20 years.

Length of protected cable: The length of cable requiring stabilization will determine the amount of concrete required and the extent of the stabilization operation (manpower, time and logistics).

CAST-IN-PLACE CONCRETE.

1. Procedure - Four basic techniques have been used in underwater concrete construction to cast concrete in place: (1) underwater bucket, (2) tremie, (3) preplaced aggregate, and (4) pumping (Waddell, 1974, and Odello, 1966). Presently, the use of cast-in-place concrete for stabilization of cables appears to be very limited, the use of the underwater bucket being the only technique cited in literature (Cullison, 1975, and Svanberg and Cox, 1973). The use of cast-in-place concrete is reported (Svanberg and Cox, 1973) to have advantages over other methods of cable stabilization when environmental conditions allow this technique to be utilized. Very little site preparation is necessary; underwater work by divers is minimized; the concrete conforms to bottom irregularities, thus increasing its holding power; and corrosion problems are reduced. One disadvantage with this technique is that retrieval of damaged cables for repair is very difficult.

Underwater bucket: An underwater bucket is a cylindrical steel container which is covered at the top with canvas or plastic sheeting to prevent mixing of the concrete with seawater. A discharge gate at the bottom, which either slides horizontally or opens like a clamshell bucket, allows the concrete to be dumped once it is lowered to the seafloor. Large buckets can handle as much as 6 to 8 yd^3 of concrete at a time (Myers et al., 1969).

Once the bucket is in place near the seafloor, the gate is opened (from the surface) and the concrete allowed to flow out over the cable. It is important that the discharge gate remain in contact with the mass of concrete on the seafloor to prevent pouring through the water column. If subsequent loads of concrete are required at the same location, it is important that the bucket land on top of the preceding lift and sink into the concrete before the gate is opened (Anon., 1974a).

Because of the extreme weight of the large buckets (as much as 17,000 pounds in water) care must be taken not to crush or damage the cable when the bucket is set on the seafloor. A large surface support platform capable of handling a 15-ton crane will also be required. Smaller buckets can be utilized, thus reducing the surface support requirements. One operation reported use of a helicopter to deliver the concrete bucket. The quality of the pours ranged from good to absolutely worthless (Cullison, 1975).

In shallow water where current or surf action may tend to wash away concrete, bucket-placed concrete is preferred because it can be a stiffer mix than concrete delivered by other techniques (Myers et al., 1969). Bucket-placed concrete should have 6 to 7 sacks of cement per cubic yard, maximum aggregate size of 1 to 2 inches, and sand content of 40% or more (Waddell, 1974). There appears to be some controversy on an acceptable slump for this technique. Waddell (1974) reports a slump of 5 to 6 inches should be used, while Svanberg and Cox (1973) proposes a maximum slump of 2-1/2 inches.

Tremie: The tremie method consists of placing concrete through a watertight vertical tube extending from above the water surface to the seafloor. To start the process, a plug consisting of either a rubber ball or a wad of burlap is placed inside the pipe below the loading hopper. Freshly mixed concrete is then fed into the hopper. This forces the plug down, displacing the seawater as it proceeds. When the plug reaches the seafloor and the pipe is completely full of concrete, the tremie is raised slightly to allow the plug to escape and the concrete to begin flowing. It is extremely important that the end of the pipe remain embedded in the newly deposited concrete to prevent its mixing with seawater and the cement's washing away. Concrete must be continuously delivered to the hopper to assure that the pipe remains full at all times. If the seal is broken while depositing, the charge will be lost, and the tremie must be withdrawn and refilled (Lorman, 1971).

Typical tremie pipes are constructed of steel, but special-purpose rubber tubes have been used occasionally. Typical sizes range from 2-1/2 to 12 inches in diameter, depending on the size of the aggregate and amount of concrete to be poured. The size of the tremie will dictate the magnitude of surface support required. For example, a 10-in.-diam tremie full of concrete will weigh approximately 100 lb/ft in seawater (Odello, 1966). The diameter will also determine the discharge rate and the capacity of the concrete mixing equipment that must be carried on the surface support platform. No optimum discharge rates have been established, but common practice indicates a linear flow of between 3/4 and 1 ft/sec to be practical (Odello, 1966). Based on this criteria, Table 4-9 lists the discharge rate for various sizes of tremie pipes.

Table 4-9. Discharge Rate Versus Diameter of Tremie Pipe

Pipe Diameter (in.)	Discharge Rate (in.-ft/min)
0.75 ft/sec Linear Flow Rate	
2	1
4	3.9
6	8.8
8	15.8
10	24.7
12	35.6
1.00 ft/sec Linear Flow Rate	
2	1.3
4	5.3
6	11.8
8	21.1
10	33
12	47.5

Concrete mixes for tremie placement must be workable and free-flowing. They are generally composed of a mix of seven sacks of cement per cubic yard, aggregate size should be less than 25% of pipe diameter, and sand content of 45% to 50%. A slump of 6 to 7 inches produces a suitable working consistency (Anon., 1974a).

Preplaced aggregate: The preplaced aggregate method utilizes a clamshell bucket or similar device to deliver coarse aggregate to the seafloor for deposit over the cable. Best results are obtainable if the aggregate can be confined within a form or steeply sloped trench (Myers et al., 1969). Grout is then pumped through tubes (which are usually installed before aggregate placement) to fill all of the voids. By starting at the bottom and allowing the grout to flow upward, the seawater is displaced, resulting in very good quality concrete with very little shrinkage (Waddell, 1974). The grout tubes may either be left in place or extracted as the grout is being pumped. The tubes should be at least 3/4 inch in diameter and spaced no more than 5 feet apart (Waddell, 1974). The grout should be pumped as soon after aggregate placement as possible to prevent silting and contamination (Myers et al., 1969).

Mixing of the grout is one of the most important aspects of this technique. Homogeneity and uniformity from batch to batch are important. A typical grout mixture contains 7 parts portland cement, 3 parts Alfesil,* 10.6 parts sand, an intrusion aid at 1% of the sum of cement and Alfesil, and enough water to obtain a flow factor of 18 to 20 seconds** (Odello, 1966).

*A proprietary pozzolantic admixture which inhibits settling, lowers water requirement, and delays setting.

**The flow factor is the amount of time required to drain a volume of 221 in.^{3} of grout from a conical cylinder having a spout 1-1/2 inches long and 1/2 inch ID.

The use of a retarding admixture requires 18 to 22 hours for the concrete to set, however grout pumping must be conducted with little or no interruption to prevent the grout tubes from becoming clogged. The proper selection of aggregate is also important in assuring proper filling of all of the voids without clogging or excessive grout pump pressure requirements. Minimum aggregate size of 3/4 inch is recommended, and maximum sizes of up to 6 inches are reported (Waddell, 1974).

Pumping: Placement of concrete by pumping is similar to tremie placement except that gravity flow is not required for distribution, thus allowing longer horizontal runs (up to 1,000 feet) and use of flexible discharge hoses. Three basic types of pumps utilized are (1) piston, (2) pneumatic or hydraulic, and (3) squeeze. These pumps vary in capacity from 10 to 75 yd^3/hr. The squeeze-type pump is generally preferred because few of the pump parts come in contact with the concrete and is easy to clean. In addition, the concrete is not subjected to extreme pressure (Lorman, 1971). This type of pump operates by squeezing the concrete through a flexible tube with a pair of rollers (Figure 4-21). As the roller pushes one charge of concrete out of the pump, a suction is created behind the roller, drawing additional concrete from the hopper.

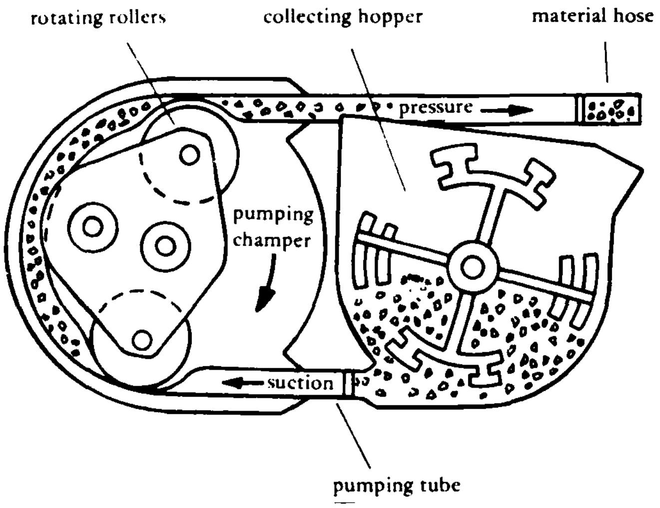


Figure 4-21. Diagram of principles involved in the squeeze-pressure type of concrete pump (patent no. 3180272).

As with all cast-in-place methods, the discharge tube must be imbedded in the previously deposited concrete to minimize mixing with seawater and the formation of laitance. When lowering the discharge line to the bottom, the end should be plugged. The pressure generated when the pump is turned on will displace the

plugs and allow the concrete to be discharged. To assure that the pipeline remains full when the pump is stopped, the end of the discharge hose should be equipped with a restricting orifice such as a gooseneck pipe. This should be designed to offer resistance to the static head while still allowing the concrete to be pumped at moderate pressures (Lorman, 1971).

An important factor in pumping concrete is the selection of aggregate, which must consist of rounded particles. Crushed rock is nearly impossible to pump since the angular particles tend to interlock. Porous aggregate should also be avoided (Lorman, 1971). Particle size is also important and should not exceed 1-1/2 inches. For best results, the gravel content should range between 55% and 58% by weight.

The concrete mix for pumping should be similar to that for tremie placement. The use of a pozzolithic admixture to increase the amount of entrained air in the concrete will improve the workability and facilitate movement of the freshly mixed concrete through the pipeline. Quantities of this type of admixture are usually specified by the manufacturer but generally average about 2 fl oz per bag of cement. The water content must be closely controlled to produce a slump of between 3 and 4 inches. Wetter mixes will tend to segregate, while dryer ones may clog the delivery tube.

2. Support requirements - Virtually no useful information on the adaptation of these methods for actual cable stabilization operations could be found. The requirements for equipment or manpower to implement these techniques are therefore difficult to assess. The anticipated cost of procuring the required equipment (which depends upon the placement method selected) is quite high. It is therefore recommended that a contractor experienced in underwater concrete construction be contacted prior to the final design and implementation of any of these methods of cable stabilization.

3. Installation time estimates - The time required to stabilize a cable with cast-in-place concrete will depend on the amount of concrete required at each site, the number of stabilization sites and distance between them, and the method selected for delivering the concrete to the seafloor. No data are available on installation times of previous stabilization operations using this technique but, because very little site preparation is required and work by divers is minimized, cast-in-place concrete should be very competitive from an installation-time standpoint.

4. Selection factors -

Bottom material and topography: This technique is applicable only on hard rock or coral seafloors. On sandy seafloors the scouring action of waves and current will tend to undermine the concrete mass, causing it to break or settle (and possibly

crush or damage the cable). Topography has little impact on this technique; however, the presence of natural trenches or ravines into which the cable can be laid and concrete poured will improve chances of a successful installation.

Waves and currents: Waves and current have a minimal effect on the concrete once it has set; this technique tends to produce a low profile mass of concrete that conforms to and locks into the seafloor. During placement of the concrete and prior to set, excessive water particle motion caused by waves and currents can have a disasterous effect on the installation. Any water motion will tend to wash the cement out of the concrete, leaving a weak crumbly mass composed primarily of aggregate. The extent of this leaching process and the effect on the concrete will depend on the water particle velocity, the amount of concrete poured, and the amount of time the concrete is exposed before it sets. Set-retarding admixtures are usually required for the methods described in this section, resulting in setting times of 12 to 18 hours. The use of accelerators would be advantageous but must be used judiciously to prevent premature setting while the concrete is still in the delivery tube or bucket. Since the delivery tube must remain within the previously delivered mass of concrete and to assure adequate equality, vertical motion of the surface craft from swells greater than about 1 foot will present significant problems in properly placing the concrete.

Logistics: The weight and bulk of the concrete make local procurement of raw materials desirable. Unless the materials are delivered as needed, an on-site area protected from moisture must be provided for storage of the cement. The requirements for mixing the concrete just prior to placement and the high delivery rate of some of the methods necessitate a large surface support craft capable of handling the bulk concrete, the mixing equipment, and the delivery hardware. Port facilities capable of transferring the premixed concrete constituents to the surface support platform should be located close to the site to prevent lengthy delays for resupply.

Weather window: This technique requires calm sea conditions (swells less than 1 to 2 feet) for a period long enough to place the concrete and allow it to set.

Visibility: Underwater visibility has little effect on the feasibility of this technique since the discharge of concrete will reduce local visibility to near zero. Initial visibility should be good enough to determine that the discharge tube or bucket is in the proper location prior to delivery of the concrete.

Hazards: Discrete masses of concrete placed at intervals along the cable provide little protection against any of the hazards discussed in Section 2.4. A continuous pour along the entire

length of the cable would be effective against anchor drag and fouling. Pumped concrete, delivered through a flexible hose positioned by divers, appears to be the only feasible method of completely encasing the cable in concrete.

Wind: Wind will affect the ability of the surface support platform to moor properly over the cable and maintain the end of the delivery tube within the mass of previously delivered concrete. The use of flexible delivery tubes positioned by divers will tend to minimize the effect of the wind on the surface support platform.

Design life: Concrete which is properly placed under water has shown good strength characteristics for periods of more than 50 years (Myers et al., 1969), which exceeds most cable system design life requirements. If the cable should become damaged (e.g., by a dragging anchor) in an area not encased in concrete it is very difficult to recover the cable for repairs.

Length of protected cable: The length of cable requiring protection will determine the amount of concrete required for stabilization and the amount of time to complete the installation operation. This technique is feasible only if the installation time for placement and set is less than the projected calm weather window.

PRECAST ELEMENTS.

1. Procedure - Precast-concrete elements may be set directly over the cable to provide direct immobilization or, more commonly, can be used in conjunction with other mass anchors such as chain (see Section 4.2.5). When set directly over the cable, the elements are generally U-shaped with sloping sides to produce a low center of gravity and legs 1 to 2 inches longer than the cable diameter to prevent crushing the cable (Webb, 1973). Rebar is usually allowed to protrude an inch or more from the bottom of the legs to increase frictional resistance to sliding along the seafloor (NMC, 1970).

The set-on weights may be poured at the job site or at a precast-concrete manufacturing plant where economy and quality can most readily be achieved (Myers et al., 1969). The elements are reinforced with steel rebar to resist transportation and handling loads. Lifting hooks are cast into the elements to facilitate deployment and positioning over the cable.

The procedure's advantage is that no setup time is required once the elements are positioned on the seafloor, and the quality of the concrete can be assured prior to deployment. However, the load-handling capability of the surface support craft will generally have to be greater than for other concrete techniques because of the size and weight of the element. Positioning of the concrete will also be more difficult if damage to the cable is to be avoided. The best procedure is to place the elements near the cable with a

crane on the surface support craft and utilize lift bags and divers for final positioning, thus eliminating the influence of swells and wind on the support craft during positioning operations.

2. Support requirements -

Manpower:* Manpower requirements will vary depending on the number of precast elements to be placed, the amount of time available to complete the operation, the water depth, and whether or not the concrete is to be cast at the work site. The following is a list of the minimum number of personnel required to support this type of operation:

Personnel	Requirements
Diving Officer	1
Divers	5
Boat Operator	1
Crane Operator	1
Riggers	2
Concrete Workers	2

Concrete workers are required only if casting and installation are to occur simultaneously.

Equipment: The only specialized equipment required for this operation is a surface support platform equipped with a crane or hoist that can handle the precast elements over the side. The following lists the minimum equipment needed for this operation.

Equipment	Requirements
1. Diving Equipment	as required
2. Concrete	as required
3. Form Material	as required

*The information provided in this section is based on limited data from a few previous installations and as such is intended only as a guide in developing preliminary cost estimates. Since environmental conditions vary considerably from site to site the final decision on the number of personnel required to safely conduct the operation must be left to the discretion of the diving officer.

	Equipment	Requirements
4.	Rebar	as required
5.	Seabee Builders Kit	1
6.	Concrete Mixer	1
7.	Surface Support Platform with Crane or Hoist	1
8.	Lift Bags	as required
9.	Compressor (LP with 200-ft hose)	1
10.	Rigging Gear (line, shackles, etc.)	as required

Items 2 through 6 are required only if concrete is to be cast on-site. But Item 6 is not needed if premixed concrete can be delivered to the site by truck.

3. Installation time estimates - Since this technique has not been employed in the past for cable stabilization, no data are available on which to base estimates of installation time requirements.

4. Selection factors - The selection parameters for precast concrete are identical to those for sacked concrete except that the concrete is set and cured before deployment thus eliminating the problem of wave and current forces washing the cement away immediately after installation.

4.2.5 Chain

Background. Chain can be an appropriate stabilization restraint for cable runs with limited life requirements or in areas of minimal hydrodynamic forces. The stabilizing capability of chain restraints can be improved by adding additional anchoring (e.g., anchor, concrete, or rock bolts) for the chain (Figure 4-22). Chain may be especially useful for stabilizing numerous cables in a confined area (Figure 4-23).

In this technique, chain is used as a convenient mass anchor. Normally, chain is readily available in the Navy system. Also, its flexible characteristics facilitate storage (Figure 4-24) and cable stabilization on irregular seafloors.

Description. Chain size is expressed in terms of wire (bar) diameter. The general overall dimensions for chain, shown in Figure 4-25 for 1-inch size, are multiples of the nominal material diameter, generally standard worldwide within the limits of inch or metric conversion and subject to applicable tolerances.

Figure 4-22. Cable stabilization using chain and concrete.

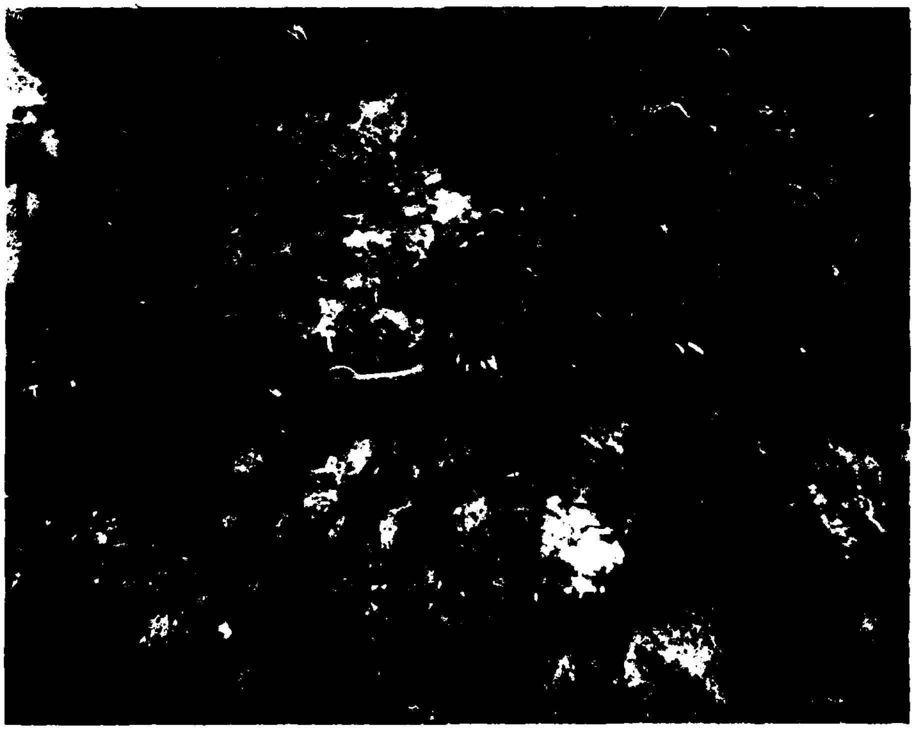


Figure 4-23. Multiple cable stabilization using chain.

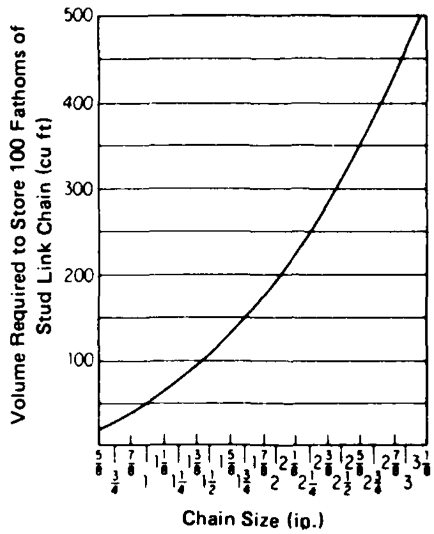


Figure 4-24. Approximate volume required for storing self-tiering stud-link chain.

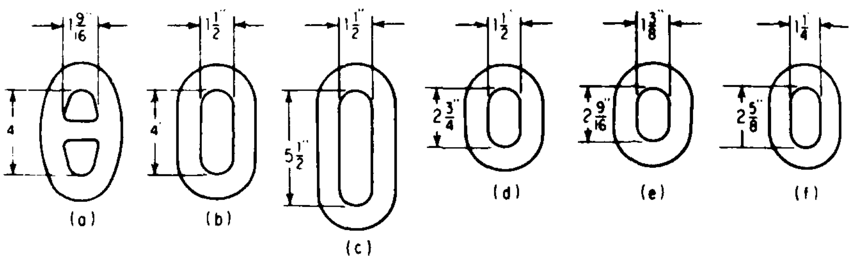


Figure 4-25. Link comparisons for 1-in. chain (left to right): stud link, buoy, long-link conveyor or topping lift, proof coil, BBB, high-test and alloy.

A wide variety of chain, up to the 3-inch size, has been used effectively for cable stabilization. Navy 2-inch anchor chain has been a popular choice. The weight and size of common chain is shown in Tables 4-10 and 4-11.

Procedure. The procedures used to anchor cables with chain are about as numerous as the number of different installations. Chain can be draped across and firmly attached to the cable, laid alongside and attached to the cable (Figure 4-26), or stretched between large anchors to provide a restraint base for multiple cables.

Often, short lengths of chain are lowered to the cable and placed in position by divers. Lift bags are often used (Figure 4-27) or the chain can be lowered on a spreader bar (a wooden 4x4 works well for 2-inch chain). In shallow water, accurate placement can be facilitated by using spar marker buoys. Since the spar buoy provides a straight line to the cable, the lowering crew can accurately determine the location of the bottom of the spar by knowing its length and measuring, or estimating, the buoy's tending angle.

Half-shots of 2-inch chain can be relatively accurately dropped from the surface in shallow water (<30 feet). The half-shot should be attached to a 4x4 spreader bar, and aiming cues should be provided by the spar-buoy, or similar, technique. Acceptable accuracy can be obtained approximately 80% of the time. If water conditions permit, a lift bag can be used to move the chain into final position. The drop technique may be the only acceptable approach to placing the chain in the surfzone. This can be done by using a LARC and a beach range marker (or other accurate navigational aid). The chain, on a spreader bar, is attached to the stern of the LARC with a pelican hook and is tripped into the water as the LARC passes the appropriate point while coming into the range.

Chain hooks are helpful in minimizing hand injuries and work well, both above and below the water. They can easily be fabricated from 1/2-inch rebar (see Figure 4-28).

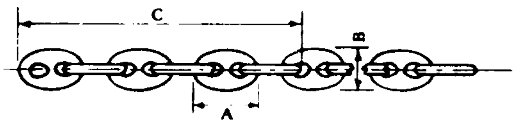
A very positive attachment should be used to fasten the cable to the chain. The fastening technique must prohibit movement of the cable against the chain.

Support Requirements.

MANPOWER.* A minimum crew of about 8 persons, including a diving supervisor and a LARC operator, would be needed for this type of operation. The operation could be facilitated with additional beach preparation and logistic support personnel.

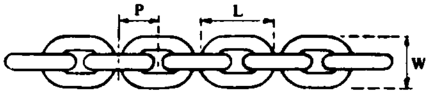
*The information provided in this section is based on limited data from a few previous installations and as such is intended only as a guide in developing preliminary cost estimates. Since environmental conditions vary considerably from site to site the final decision on the number of personnel required to safely conduct the operation must be left to the discretion of the diving officer.

Table 4-10 Standard Stud Link Chain Dimensions



Chain Size		Link Length, A (in.)	Link Width, B (in.)	Length Over Six Links, C (in.)	Approximate Weight Per 15-Fathom Shot (lb)
in.	mm				
3/4	19.05	4-1/2	2-5/8	19-1/2	505
13/16	20.64	4-7/8	2-7/8	21-1/8	600
7/8	22.23	5-1/4	3-1/8	22-3/4	688
15/16	23.81	5-5/8	3-5/16	24-3/8	795
1	25.40	6	3-9/16	26	900
1-1/16	26.99	6-3/8	3-3/4	27-5/8	1,020
1-1/8	28.58	6-3/4	4	29-1/4	1,140
1-3/16	30.16	7-1/8	4-1/4	30-7/8	1,275
1-1/4	31.75	7-1/2	4-1/2	32-1/2	1,415
1-5/16	33.34	7-7/8	4-3/4	34-1/8	1,560
1-3/8	34.93	8-1/4	4-15/16	35-3/4	1,705
1-7/16	36.51	8-5/8	5-3/16	37-3/8	1,865
1-1/2	38.10	9	5-3/8	39	2,035
1-9/16	39.69	9-3/8	5-5/8	40-5/8	2,195
1-5/8	41.28	9-3/4	5-7/8	42-1/4	2,345
1-11/16	42.86	10-1/8	6-1/16	43-7/8	2,530
1-3/4	44.45	10-1/2	6-5/16	45-1/2	2,720
1-13/16	46.04	10-7/8	6-1/2	47-1/8	2,925
1-7/8	47.63	11-1/4	6-3/4	48-3/4	3,125
1-15/16	48.21	11-5/8	7	50-3/8	3,335
2	50.80	12	7-3/16	52	3,525
2-1/16	52.39	12-3/8	7-7/16	53-5/8	3,750
2-1/8	53.98	12-3/4	7-5/8	55-1/4	3,975
2-3/16	55.56	13-1/8	7-7/8	56-7/8	4,215
2-1/4	57.15	13-1/2	8-1/8	58-1/2	4,460
2-5/16	58.74	13-7/8	8-5/16	60-1/8	4,710
2-3/8	60.33	14-1/4	8-9/16	61-3/4	4,960
2-7/16	61.91	14-5/8	8-3/4	63-3/8	5,210
2-1/2	63.50	15	9	65	5,528
2-9/16	65.09	15-3/8	9-1/4	66-5/8	5,810
2-5/8	66.68	15-3/4	9-7/16	68-1/4	6,105
2-11/16	68.26	16-1/8	9-11/16	69-7/8	6,410
2-3/4	69.85	16-1/2	9-7/8	71-1/2	6,725
2-13/16	71.44	16-7/8	10-1/8	73-1/8	7,040
2-7/8	73.03	17-1/4	10-3/8	74-3/4	7,365
2-15/16	74.61	17-5/8	10-9/16	76-3/8	7,696
3	76.20	18	10-13/16	78	8,035
3-1/16	77.79	18-3/8	11	79-5/8	8,379
3-1/8	79.38	18-3/4	11-1/4	81-1/4	8,736
3-1/16	80.96	19-1/8	11-1/2	82-7/8	9,093
3-1/4	82.55	19-1/2	11-11/16	84-1/2	9,460
3-5/16	84.14	19-7/8	11-15/16	86-1/8	9,828
3-3/8	85.73	20-1/4	12-1/8	87-3/4	10,210
3-7/16	87.31	20-5/8	12-3/8	89-3/8	10,599
3-1/2	88.90	21	12-5/8	91	10,998
3-5/8	92.08	21-3/4	13-1/16	94-1/4	11,607
3-3/4	95.25	22-1/2	13-1/2	97-1/2	12,626
3-7/8	98.43	23-1/4	14	100-3/4	13,340
4	101.60	24	14-3/8	104	14,100
4-1/8	104.78	24-3/4	14-7/8	107-1/4	15,000
4-1/4	107.95	25-1/2	15-5/16	110-1/2	15,900
4-3/8	111.13	26-1/4	15-3/4	113-3/4	16,860
4-1/2	114.30	27	16-3/16	117	17,840
4-5/8	117.48	27-3/4	16-5/8	120-1/4	18,840
4-3/4	120.65	28-1/2	17-1/8	123-1/2	19,840

Table 4-11. Close-Link Hoisting, Sling, and Crane Chain



Size (in.)	Standard Pitch, P	Average Weight Per Foot (lb)	Outside Length, L (in.)	Outside Width, W (in.)
1/4	25/32	3/4	1-5/16	7/8
5/16	27/32	1	1-1/2	1-1/16
3/8	31/32	1-1/2	1-3/4	1-1/4
7/16	1-5/32	2	2-1/16	1-3/8
1/2	1-11/32	2-1/2	2-3/8	1-11/16
9/16	1-15/32	3-1/4	2-5/8	1-7/8
5/8	1-23/32	4	3	2-1/16
11/16	1-13/16	5	3-1/4	2-1/4
3/4	1-15/16	6-1/4	3-1/2	2-1/2
13/16	2-1/16	7	3-3/4	2-11/16
7/8	2-3/16	8	4	2-7/8
15/16	2-7/16	9	4-3/8	3-1/16
1	2-1/2	10	4-5/8	3-1/4
1-1/16	2-5/8	12	4-7/8	3-5/16
1-1/8	2-3/4	13	5-1/8	3-3/4
1-3/16	3-1/16	14-1/2	5-9/16	3-7/8
1-1/4	3-1/8	16	5-3/4	4-1/8
1-5/16	3-3/8	17-1/2	6-1/8	4-1/4
1-3/8	3-9/16	19	6-7/16	4-9/16
1-7/16	3-11/16	21-1/2	6-11/16	4-3/4
1-1/2	3-7/8	23	7	5
1-9/16	4	25	7-3/8	5-5/16
1-5/8	4-1/4	28	7-3/4	5-1/2
1-11/16	4-1/2	30	8-1/8	5-11/16
1-3/4	4-3/4	31	8-1/2	5-7/8
1-13/16	5	33	8-7/8	6-1/16
1-7/8	5-1/4	35	9-1/4	6-3/8
1-15/16	5-1/2	38	9-5/8	6-9/16
2	5-3/4	40	10	6-3/4
2-1/16	6	43	10-3/8	6-15/16
2-1/8	6-1/4	47	10-3/4	7-1/8
2-3/16	6-1/2	50	11-1/8	7-5/16
2-1/4	6-3/4	53	11-1/2	7-5/8
2-3/8	6-7/8	58-1/2	11-7/8	8
2-1/2	7	65	12-1/4	8-3/8
2-5/8	7-1/8	70	12-5/8	8-3/4
2-3/4	7-1/4	73	13	9-1/8
2-7/8	7-1/2	76	13-1/2	9-1/2
3	7-3/4	86	14	9-7/8

Figure 4-26. Attachment of cable to chain stabilization.

Figure 4-27. Divers utilizing lift bags to transport and position chain sections.

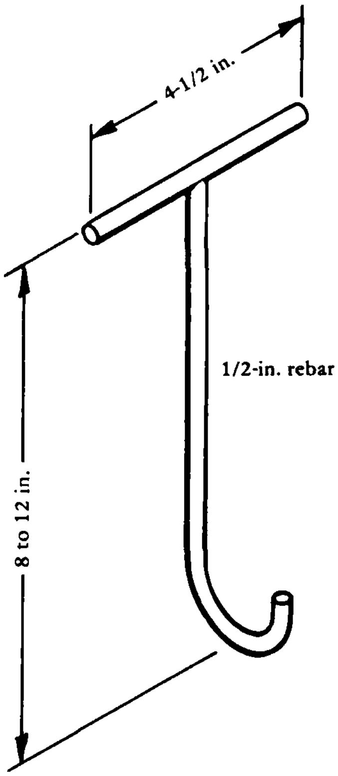


Figure 4-28. Chain hook.

EQUIPMENT. In addition to appropriate diving gear, this type of operation would need a LARC fitted with a stiff-leg crane if it is to be supported from the beach or if surfzone applications are needed. Otherwise any craft capable of handling a half-shot of 2-inch chain could be used. Miscellaneous line, 4x4 spreader bar material, lift bags, pelican hooks, a zodiac with 25-hp outboard, and miscellaneous rigging materials should also be on the mount-out list of materials.

Installation Time Estimates. Beach preparations and staging will take from 1 to 2 days, depending on the size and nature of the operation. Using a LARC V, five shots of chain can be placed over a cable, including one in the surfzone, and attached in approximately 1 day under good conditions.

Selection Factors.

BOTTOM MATERIAL AND TOPOGRAPHY. The chain stabilization technique may be used on any type of bottom material or bathometry if the cable is exposed for attachment at the time of installation.

WAVES. Surf less than 3 feet and swells less than 6 feet are needed for installation.

CURRENT. Currents or surges exceeding 1 knot would make lowering the chain and fastening to the cable very difficult for the divers.

LOGISTICS. Adequate logistics would be necessary to place the equipment, material, and personnel on the scene. The operation could be supported from sea. If the operation is to be supported from the beach or if surfzone application is necessary then a LARC is required. Chain is heavy, about 1,200 lb per shot; thus requiring heavy-load handling equipment on-site.

VISIBILITY. Although it is desirable to have 3 feet or greater visibility for attaching operations, with proper planning these evolutions can be conducted in zero visibility (but with longer time requirements).

HAZARDS. Chain does not appreciably improve a cable's capability of withstanding ice scour, marine organism attack, or anchor/ trawler drags. When properly installed chain will improve the cable's resistance to abrasion.

WIND. It would be desirable to have wind of <15 to 20 knots for at-sea chain emplacement and diving.

LENGTH OF PROTECTED CABLE. This process can be used on almost any length of cable if time and resources permit.

4.3 TIE-DOWNS

Tie-downs are immobilization techniques which mechanically couple the cable to the seafloor at discrete points along the length of the cable. This type of protection technique is generally used when the cable or cable/mass anchor system is not sufficiently heavy to remain stable under the influence of maximum anticipated hydrodynamic loads. Most of the design theory (Chapter 6) deals with this type of protection technique.

The techniques presented in this section include:

(1) Pin anchors

(2) Grouted fasteners

(3) Rockbolts

4.3.1 Pin Anchors

Background. Pin-type anchors have been used in both soft and hard seafloors to stabilize cables and pipelines. In the past, the pin anchor used to stabilize a 5-in.-diam cable on coral consisted of a 1-in.-diam steel bar 3 ft long with an L-shaped clip at the top (Figure 4-29). The pin was jackhammered into the coral until the clip was secured over the cable. This type of pin installation worked satisfactorily, but after about 3 years corrosion of the steel caused sufficient reduction of friction between the pin and sides of the hole that the pins began to pull out. Development of seafloor rock-bolting equipment and techniques (Section 4.3.3) has made the driven pin anchor obsolete for coral seafloors and further discussion in this handbook is omitted.

Pin anchors for sand or clay seafloor materials have been used primarily to stabilize large-diameter pipelines which are only slightly negatively buoyant. Although the application differs slightly, adaptation of this technique to cable stabilization would utilize similar techniques and equipment. The design engineer must be cautious about applying this technique in areas where self-burial of the cable would tend to separate the cable from the anchor or in areas where significant sand transport exists. This condition could either leave the cable or anchor suspended above the bottom or even dislodge the anchor system completely.

Since cables are generally much smaller in diameter than pipelines, the spacing between auger pins used in the same saddle needs investigation to assure that they don't interact, thus causing a significant reduction in holding capacity.

Description. Sand and clay anchors work on an auger principle. They consist of a rod with a disk which is shaped in the form of a screw (Figure 4-30). Installation is accomplished by turning the rod clockwise which draws the anchor down into the seafloor (Webb, 1973).

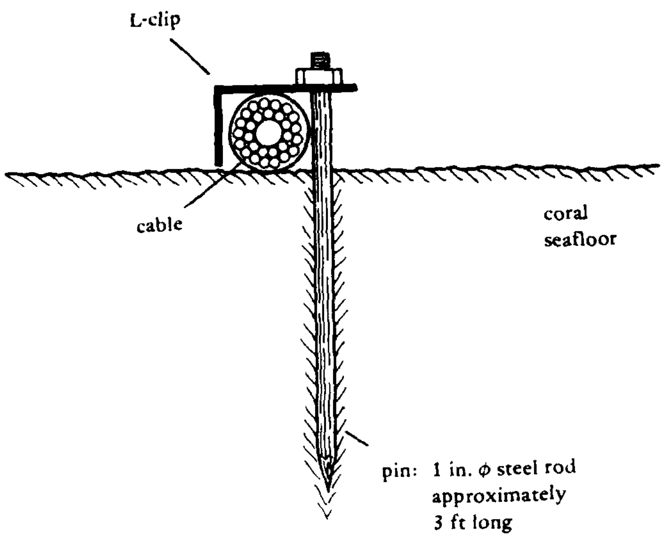


Figure 4-29. Driven pin anchor.

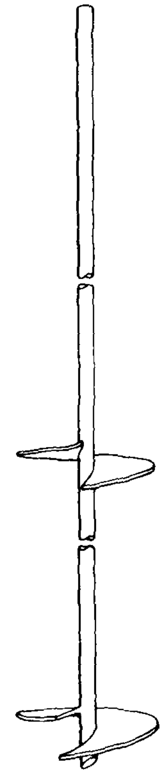


Figure 4-30. Auger pin anchor.

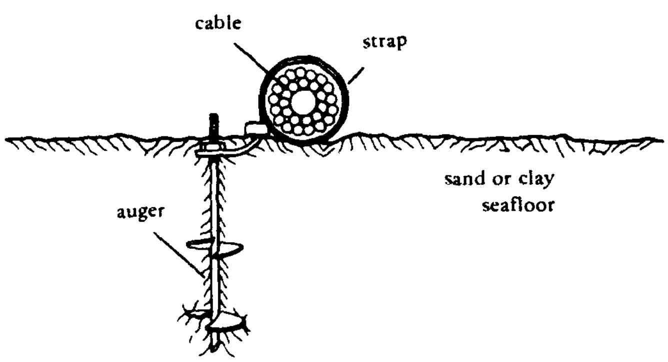


Figure 4-31. Strap connection.

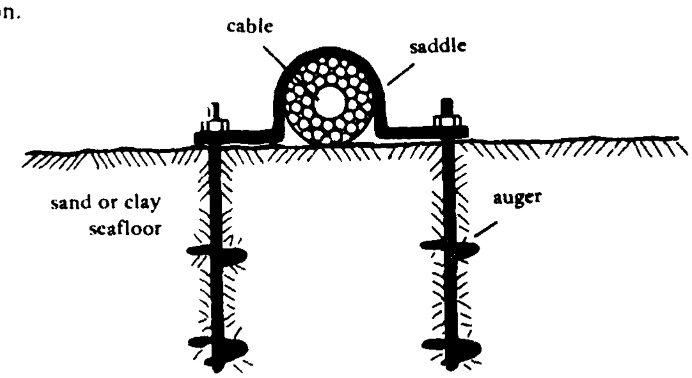


Figure 4-32. Saddle connection.

Augers with multiple helix plates and rod lengths up to 12 feet have been used where hydrodynamic forces are expected to be large and the seafloor soil is weak.

Numerous theories have been developed to aid in the design of auger-type anchor construction and installation, but most have proven to be limited to a particular location and soil type or are inadequate altogether. Because of the lack of appropriate design theory, successful stabilization system design can be assured only if local seafloor soil properties are determined and on-site anchor pullout tests are conducted. The only universally accepted rule-of-thumb is that the uppermost helix should be buried to a depth >5 diameters of the helix (Short, 1967).

The auger is secured to the cable with either a strap which encircles the cable and attaches to the auger (Figure 4-31), or the augers are installed in pairs with a U-shaped saddle straddling the cable (Figure 4-32).

The capacity of any given installation will be dependent on the diameter of the helix, the depth of burial, and the properties of the seafloor soil. Holding capacity of these anchors of 5,000 to 26,000 lb per anchor have been reported.

Procedure. Auger pins are usually imbedded with an installation unit consisting of a buoyancy package, a torque motor (either pneumatically or hydraulically operated from the surface), and a framework to hold the components and provide the reaction torque required for installation. The auger and strap or saddle are positioned in the installation unit on-board the surface support craft and then lowered to the seafloor where it is positioned over the cable by divers. The torque motor is then actuated and the auger rotated into the seafloor until the saddle comes in contact with the cable. The installation unit is retrieved and a new auger set installed.

Support Requirements.

MANPOWER.* No specialized personnel are required to implement this type of operation. The number of diving personnel required will depend on the water depth, the number of anchors to be installed, and the length of time allowed for completion. A minimum crew of 10 consisting of the following should be on-site for this type of operation.

*The information provided in this section is based on limited data from a few previous installations and as such is intended only as a guide in developing preliminary cost estimates. Since environmental conditions vary considerably from site to site the final decision on the number of personnel required to safely conduct the operation must be left to the discretion of the diving officer.

Personnel	Requirements
Diving Officer	1
Divers	5
Boat Operator	1
Hoist Operator	1
Riggers	2

EQUIPMENT. Commercially available installation units typically weigh about 6,000 lb, and this necessitates a surface support craft equipped with a crane or hoist capable of handling at least 3 tons over the side. The following lists the major hardware and equipment required to support this type of operation.

Equipment	Requirements
Surface Support Craft with 3-ton Crane	1
Diving Gear	as required
Augers	as required
Straps and Saddles	as required
Hydraulic Power Source	1
Compressor (LP)	1
Hydraulic Hose (3/4-in. diam, 250-ft length)	2
Pneumatic Hose (1-in. diam, 250-ft length)	1
Rigging Gear	as required

Installation Time Estimates. Commercial firms, specializing in pipeline stabilization, report installation times of 5 to 10 minutes per set of two anchors. It is doubtful that this includes the time for lowering, positioning, retrieving, or moving the barge in its moorings.

Selection Factors.

BOTTOM MATERIAL AND TOPOGRAPHY. The auger pin anchor is applicable only in sand or clay seafloors. The properties of the seafloor material must be known in order to adequately design the auger in terms of diameter and length. Seafloors with

radically changing topography should be avoided because of the danger of leaving the cable suspended above the seafloor or even completely dislodging the anchor.

WAVES. Surge from waves will affect the divers' ability to properly position the installation unit. Surge of greater than 1-1/2 knots will most likely make this technique unfeasible for equipment handling. The effect of swells greater than 2 feet high on the surface support platform during the lowering and lifting of the installation unit could be dangerous to the equipment and the diver unless some type of motion-compensation load-handling equipment is used.

CURRENT. Current velocities in excess of 1-1/2 knots will create problems for the diver trying to position the installation unit.

LOGISTICS. A surface support craft equipped with a 3-ton capacity crane or hoist and adequate space for divers and power sources will be required to support this operation.

WEATHER WINDOW. A calm weather window at least 4 hours each day is required if any useful work is to be accomplished. Adverse weather immediately after installtion of the auger pins should have no adverse effects if they are properly designed and installed.

VISIBILITY. Underwater visibility >15 feet will facilitate positioning of the installation unit. However, with calm seasurface conditions where the installation unit will not be subject to abrupt motions, this operation can be safely conducted with visibility as low as 5 to 6 feet.

HAZARDS. The use of auger pins will provide very little, if any, protection for the cable against any of the hazards discussed in Section 2.4. On a stable seafloor, this technique will provide protection against hydrodynamic loads.

WIND. Wind will affect the ability of the surface support craft to remain moored directly over the cable stabilization site. Winds greater than 20 knots generally make diving operations difficult and often hazardous.

DESIGN LIFE. Since this technique has not been widely used for stabilization of cables, accurate estimates of reliable design life are difficult to make. It is typical for a pipeline stabilization system of this type to be equipped with zinc anodes sufficient to provide 18 to 20 years of cathodic protection (Anon., 1971), indicating an anticipated operational life of at least 15 to 20 years.

LENGTH OF PROTECTED CABLE. The length of cable requiring protection has little effect on the technical feasibility of this technique unless the operation cannot be completed within the available weather window.

4.3.2 Grouted Fasteners

Background. In high-surge areas, split-pipe-covered cable laying on rocky bottoms have been immobilized by grouted-in U-shaped fasteners (U-rods). Recently, rockbolts have been taking the place of some or all of the normally used U-rods. Rockbolts are discussed in Section 4.3.3.

Properly installed U-rods provide a formidable immobilization technique but require fairly good diving conditions and considerable time. Roughly four to six U-rods can be installed per 8-hour day in good conditions. This is costly in time when one considers that a U-rod may have to be spaced about every third split-pipe section (9 feet). Although the cost of the U-rods (and ancillary supplies and gear) is small compared to project costs, it is not insignificant at $40 (1974) per copper-nickel* (CuNi) U-rod.

The U-rods must be installed tight against the top of the split pipe. If not tight, then the pipe will move; the associated abrasion, wear, and vibration may ultimately lead to cable failure (Figure 4-33). This problem was one of the primary motivators in replacing U-rods with rockbolts since the rockbolt automatically forces the split-pipe

Figure 4-33. Cable failure due to abrasion on improperly installed U-rod.

*CuNi was specified by marine metallurgists as the best choice for long-term installations.

tightly against the rock. In some weak seafloor rock, however, grouted fasteners such as U-rods can develop holding strength considerably greater than expansion anchors (i.e., rockbolts).

Description. U-rods are U-shaped staples (Figure 4-34) fabricated either from 1-in.-diam 316 stainless steel rod stock or from 1-1/8-in.-diam CuNi rod stock. The clearance inside the U is set to just clear the maximum circumferential projections of the type of split-pipe used, and the length of the legs should provide at least a 10-in. insertion into the grout (or bottom). It should be noted that bottoms can be extremely irregular causing a less than desirable insertion length (Figure 4-35).

Notches or grooves were originally cut in the bottom of the legs of the U-rod to improve pullout resistance; however, it has been found that threading the last 3 inches of each leg, and installing a nut (before the U-rod is grouted) provides the same pullout resistance at a much lower fabrication cost. The U-rods are inserted into 2-1/2- or 3-in.-diam predrilled holes and secured with a hydraulic cement similar to Waterplug, which contains an admixture to promote rapid setting. In warm environments (>70-degree water temperature), initial set may take place in as little as 3 or 4 minutes after mixing with water.

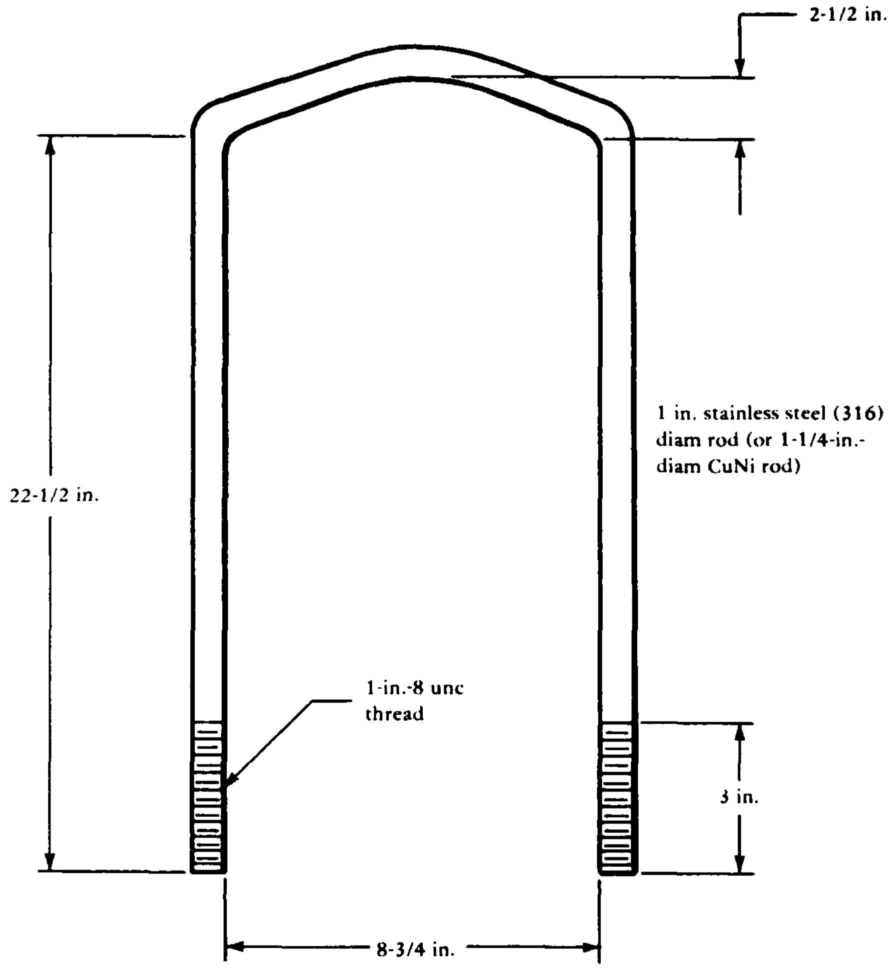
Results of tests reported by Parisi and Brackett (1974) concluded that in hard, competent rock, the diameter of the grout hole has no effect on the pullout strength of the grouted fastener.* A minimum annular clearance of at least 3/8 in. around the grouted fastener is recommended, however, to assure proper distribution of the grout.

Recent experiments conducted by Thatcher (1977) using epoxy resin grout to secure bolts in coral under water showed exceptionally good holding strengths as long as the epoxy was mixed on the surface and placed in the hole so that seawater mixing was minimized. Table 4-12 indicates average holding capacities of various grouting compounds.

Procedure. A drill template is normally used to get proper spacing and alignment of U-rod holes. A typical template is illustrated in Figure 4-36. Rock drilling with a pneumatic sinker drill is an operation difficult for any diver; percussion effects of the drill produce discomfort (at the least) in all divers and nausea and dizziness in some divers. If a diver can, keeping the head and body away from the top of the drill helps. A hydraulic rotary impact drill has been developed which eliminates the percussion problems associated with the pneumatic drills. However, only four of these drills have been fabricated to date (Brackett and Tausig, 1977).

The drill rod should be clearly marked so that the diver can easily determine when the hole is deep enough. The hole depth must be measured from the top of the split-pipe, not the depth in rock. It should be understood by the drill operators that if the hole is not deep enough, the U-rod will not fit down snug over the split-pipe; experience has shown that this is as good as no U-rod at all.

*Similar tests in weak rock and coral have not yet been conducted.



NOTE: 1-in.-8 unc nut to be threaded on bottom of each leg prior to installation to improve bonding to grout.

Figure 4-34. U-rod configuration.

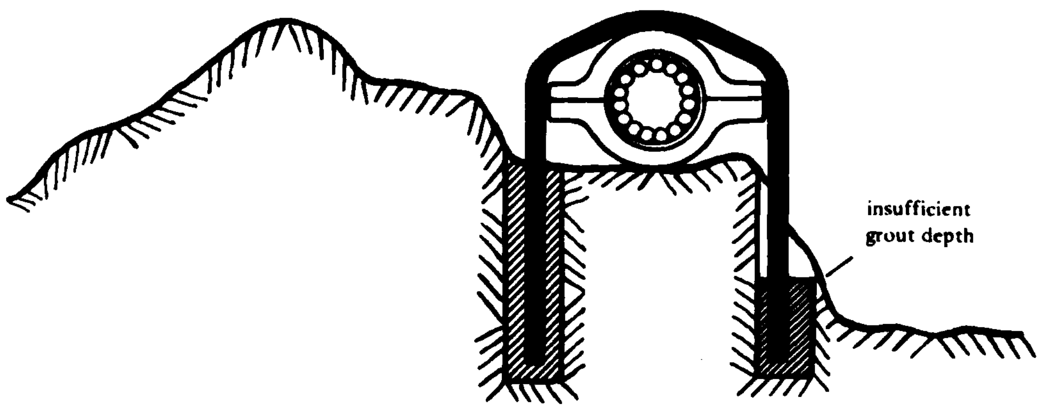


Figure 4-35. Improper U-rod installation.

Table 4-12. Average Holding Capacity of Grouting Compounds

Grout Type	Mixing Environment	Test Environment	Average Holding Capacity (lb)
Hydraulic Cement	Underwater	Underwater	5,000
Hydraulic Cement	Air	Land	37,800
Epoxy Resin	Underwater	Underwater	0 to 700
Epoxy Resin	Air	Underwater	36,700

It is recommended that the U-rods be grouted in place shortly after they are drilled because: (1) the holes can become silted-in quite rapidly, (2) the holes can be difficult to relocate, and (3) the split-piped cable can (and does) shift.

In previous U-rod installations a toothpaste-tube technique was used to dispense the grout. Recently, however, an underwater hydraulic-powered grout pump has been developed that may speed the operation considerably (Parisi and Brackett, 1974).

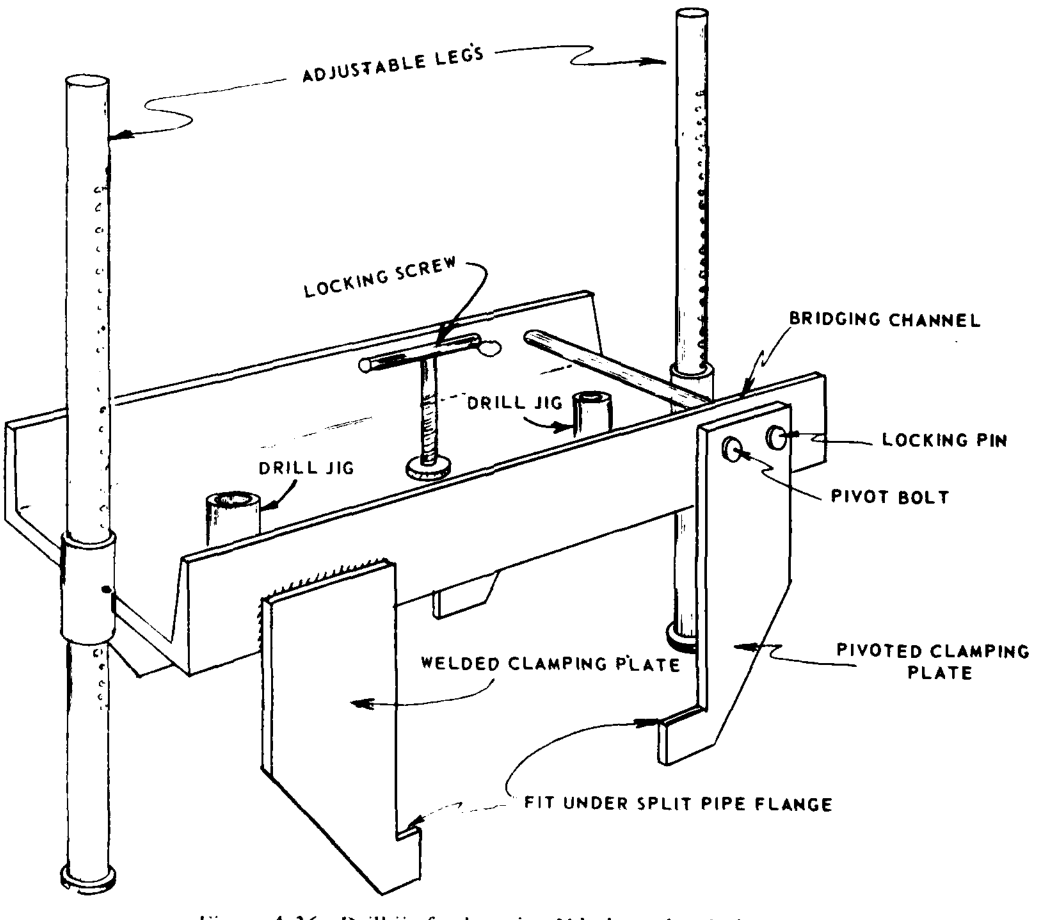


Figure 4-36. Drill jig for locating U-bolt anchor holes.

The toothpaste-tube technique employs polyethylene plastic tubing for mixing and dispensing the grout. Tubing 5 in. in diameter and 6-mil thick is recommended. Too small a diameter makes mixing very difficult, while tubing too large is difficult for the diver to handle. A 4- to 5-ft length of tubing is cut off and twisted tightly (at least 4 complete twists) in the middle. Then the proper proportions of a hydraulic cement and water* are placed in each side and the ends tied off. This produces a ready-to-go system that can be mixed and used when called for. For Waterplug cement the proper mixing proportions are one part water to three parts cement (by volume); 3 quarts of cement are required to fill each hole.

The time required for the hydraulic cement to set depends on the type of cement and the temperature. In tropical conditions, the diver may actually have to take the unmixed tube of cement and water down to the work site and mix them there just before insertion. On a previous operation, a technique was employed that was almost as fast as bottom mixing, produced better mixing, and saved the divers from the ascent/descent cycle (a procedure which can produce delays in finding the holes, especially in dirty water). Using line-pull signals or diver communication, the divers call for two tubes of grout. Surface personnel then mix the grout and slide the tubes down a descending line to the divers, who extrude it into the hole.

If setup time isn't a problem, good results have been attained by squeezing the grout in the hole and then inserting the U-rod. If the grout is starting to get tacky, it may be best to squeeze the grout in around the already inserted U-rod.

Insertion of the grout is an important task, and the divers should be allowed some experimentation and practice if possible. In extruding the grout, it is a good idea to start from the bottom of the hole and slowly pull out the tube as the grout is squeezed out (to minimize voids and mixing with seawater). To do this, the 5-in. diam tube will have to be "rolled" or squeezed down to fit inside the 2-1/2- to 3-in. diam hole without pinching the tube off.

Once the grout is in the hole, the U-rod must be down firmly against the top of the split-pipe. If the grout is starting to get tacky, a small sledge can be helpful. In areas where any surge or current action exist, it is a good idea to wrap and weight the plastic tube material around the U-rod on top of the grout to prevent the water action from washing the top layer of grout away. A water-resistant putty material can also be used for this operation.

Design Considerations. Since U-rods cannot be pretensioned like rockbolts to provide a clamping force on the cable/split-pipe, the design equations contained in Chapter 6 must be modified. For calculating the required spacing between U-rods, the value of T_B (bolt pretension) must be set equal to zero in these equations.

*The water should be fresh since saltwater reduces the strength of the grout.

Support Requirements.

MANPOWER.* In addition to boat operators, a diving team of at least 9 men is needed to efficiently conduct this type of operation. If environmental conditions reduce bottom times, then more divers may be needed to facilitate operations.

EQUIPMENT. A surface craft must be provided which will support all diving station equipment plus a 300 SCFM air compressor if pneumatic drills are used or a 10 gpm hydraulic power source for the hydraulic rockdrill. A larger compressor may be required if the water depth exceeds 70 ft or more than one pneumatic drill is to be used at one time. Each rock drill needs approximately 100 ACFM and 90 psi above ambient at the tool. At least two rock drills with spare parts should be provided.

Installation Time Estimates. Installation times can vary greatly, depending on conditions, type of rock, equipment, and experience. At the beginning of an operation, it would probably be best to plan on installing only about 4 to 6 sets of U-rods/day/crew; this rate may double with experience.

Selection Factors.

BOTTOM MATERIAL AND TOPOGRAPHY. This technique is only required on rocky and coral bottoms where high surge conditions are expected.

WAVES. For the installation, at-sea conditions must be amenable to surface-supported diving operations.

CURRENT. Bottom currents or surges greater than 1 knot can produce problems for the underwater worker.

VISIBILITY. Underwater visibility will definitely affect the operation. While operations can be conducted in zero-visibility conditions, the time requirements will increase considerably. Fortunately, good visibility is usually found in rocky areas that require this procedure.

HAZARDS. U-rods greatly improve a cable's ability to withstand the hydrodynamic forces from waves and current. The ability to withstand anchor drag will depend upon accurately predicting the magnitude of these forces at the particular site and designing the system to withstand them. U-rod immobilization

*The information provided in this section is based on limited data from a few previous installations and as such is intended only as a guide in developing preliminary cost estimates. Since environmental conditions vary considerably from site to site the final decision on the number of personnel required to safely conduct the operation must be left to the discretion of the diving officer.

provides little, if any, protection against ice scoring and trawler drags; however, the latter is usually not associated with rocky seafloor areas.

WIND. Winds greater than 20 knots make diving operations difficult, especially if the wind is acting over a significant fetch.

DESIGN LIFE. With proper selection of materials and identification of realistic design loads, U-rod immobilization systems can be designed and installed to meet virtually any design-life requirement.

LENGTH OF PROTECTED CABLE. The length of protected cable has very little impact on the feasibility of U-rod stabilization; however, for long cable immobilization operations, sufficient personnel must be available to assure its completion within the weather window.

4.3.3 Rockbolts

Background and Description. For many years the construction industry has used expansion-type rockbolts, but it was not until 1972 that these bolts were used extensively for stabilization of seafloor cables. Subsequently, CEL conducted test and evaluation of commercially available rockbolts for seafloor fastening, and a special rockbolt was developed for use in soft seafloor coral formations (Brackett and *Parisi*, 1975).

All nongrouted rockbolts utilize the same principle to develop their anchoring strength: by mechanically expanding the downhole end of the bolt, an anchoring force is obtained through a combination of friction, adhesion between the anchor and rock (Lang, 1961), and physical penetration of the anchor into the rock.

Although rockbolts are commercially available in a variety of forms, they can generally be classified into two types: (1) drive-set and (2) torque-set.

DRIVE-SET FASTENER. The slot-and-wedge bolt (Figure 4-37) and the cone-and-stud anchor (Figure 4-38) are common examples of the drive-set fastener. The slot-and-wedge fastener is secured by placing the wedge into the slot and positioning the rod into the pre-drilled hole. The anchor is secured by driving the slotted rod over the wedge, which rests on the bottom of the hole and causes the rod to expand into the rock.

Successful installation of the drive-set type of fastener depends on accurately drilling the hole to a predetermined depth and on applying sufficient force to completely expand the slotted rod. Problems can be encountered in soft rock where the driving force pushes the wedge into the rock rather than the wedge expanding the anchor.

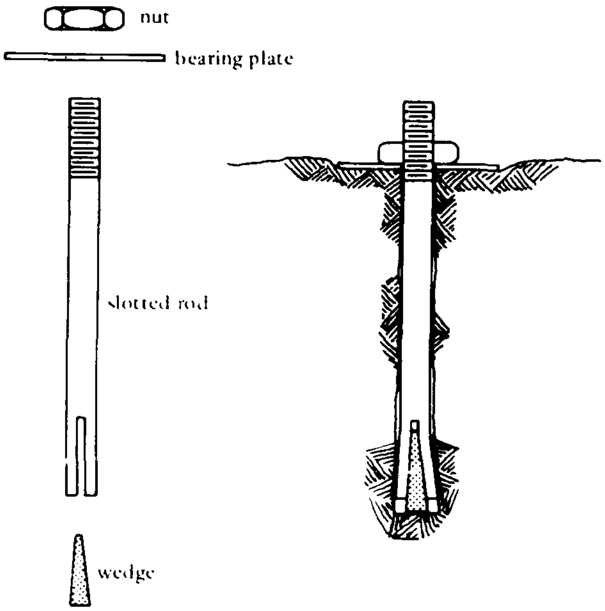


Figure 4-37. Slot-and-wedge bolt.

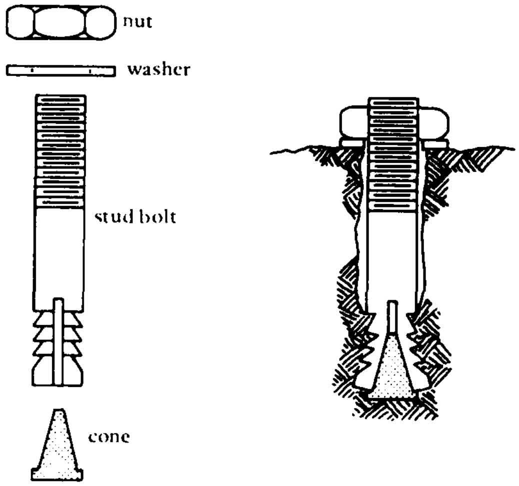


Figure 4-38. Cone-and-stud anchor.

TORQUE-SET FASTENER. A typical torque-set anchor is shown in Figure 4-39. This type of rock anchor has a wedge or cone threaded to the bottom of the bolt. A sleeve or shell that is pushed into the hole with the bolt surrounds the cone. Once the bolt has been inserted, torque applied to the nut pulls the bolt and cone up through the sleeve, thus securing the anchorage.

The torque-set type of bolt requires far less precision in hole drilling as long as the depth of the hole is greater than the length of the bolt. Expansion of the anchor is also unaffected by the quality of the rock at the bottom of the hole. With the hand and hydraulically powered tools currently available for underwater construction and salvage, it is easier to provide the required torque for the torque-set type of anchor than the required linear impact for the drive-set fastener.

Commercially available torque-set rockbolts can be obtained in sizes ranging from 1/4 to 2-1/2 in. in diameter and up to 2 ft long. Two basic configurations have been utilized to date: (1) a masonry stud bolt (Figure 4-40) and (2) a mine tunnel roof bolt (Figure 4-41). These bolts differ primarily in the ratio of shank size to anchor size and the method of expanding the anchor. Masonry stud bolts have a shank diameter equal to the anchor diameter while the mine tunnel bolt utilizes a shank slightly less than one-half the diameter of the anchor. The masonry stud bolts are best-suited for hard rocks (compressive strengths greater than 7,000 psi) while the mine tunnel bolts work best in softer rock where the larger anchor contact area reduces the local crushing stress exerted on the rock when the bolt is loaded.

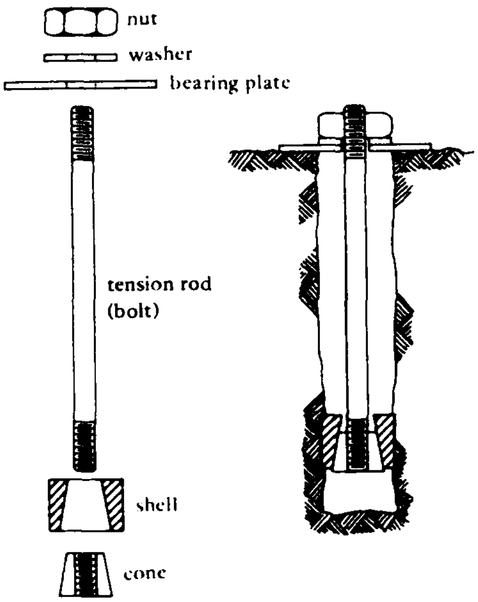


Figure 4-39. Typical torque-set anchor.

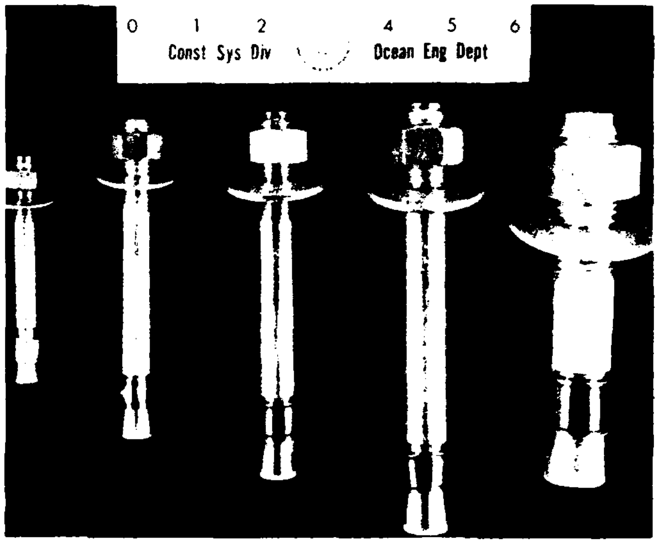


Figure 4-40. Masonry stud anchor.

For very weak coral formations a special bolt was developed for stabilizing cables (Figure 4-42). The anchor portion of the bolt consists of four separate fingers held together at the top with a collar. An internal ledge assures that the cone will not pull through the shell once maximum expansion is obtained. A slot near the top of each finger assures uniform expansion and reduces bending stresses (Figure 4-43).

Installation Procedure. Installation of a masonry stud bolt begins with the drilling operation. The hole must be the same diameter as the bolt and at least as deep as the bolt is long. The bolt is inserted in the hole anchor end first and driven down with a hammer until the washer and nut meet the surface of the rock. To prevent damage to the threads during the driving operation, the nut should be run up so that the top of the nut is slightly above the top of the bolt. Tightening the nut sets the bolt by drawing the conical portion of the bolt up through the stationary wedges, thus forcing them into the surrounding rock.

The mine tunnel bolt utilizes a cone and expanding shell similar in principle to the stud anchor; however, the mechanism for setting the anchor varies slightly. The low portion of the load bearing rod is threaded into the cone portion of the anchor. After the hole is drilled and the rockbolt inserted, the rod is rotated to draw the cone up into the collar, thus causing the collar to expand into the rock. The collar is prevented from moving up along the rod by a thrust ring located immediately above the anchor. Installation of the coral rockbolt is identical to the mine tunnel bolt.

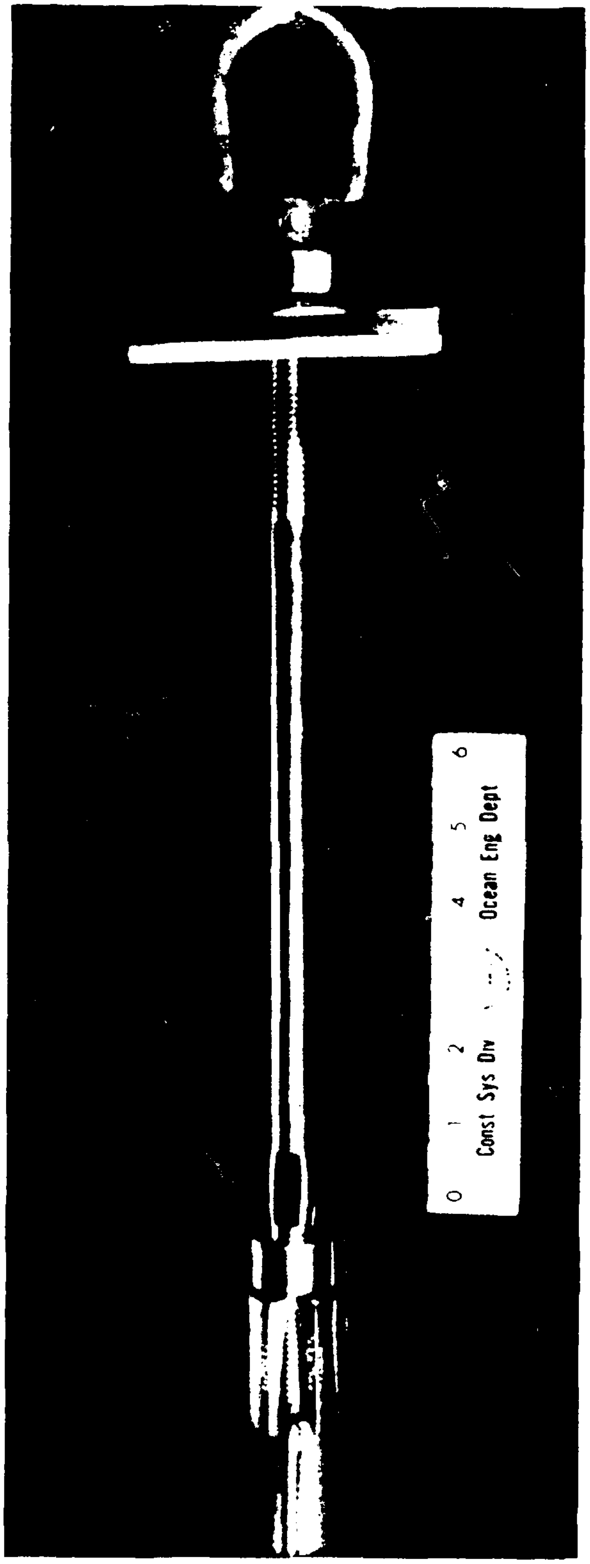


Figure 4-41. Mine tunnel bolt.

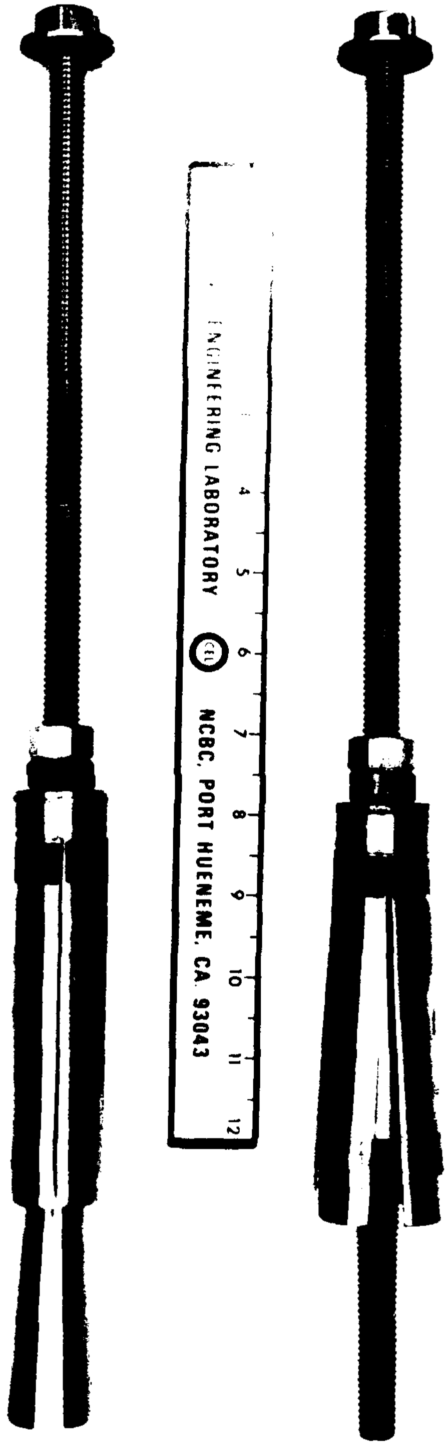
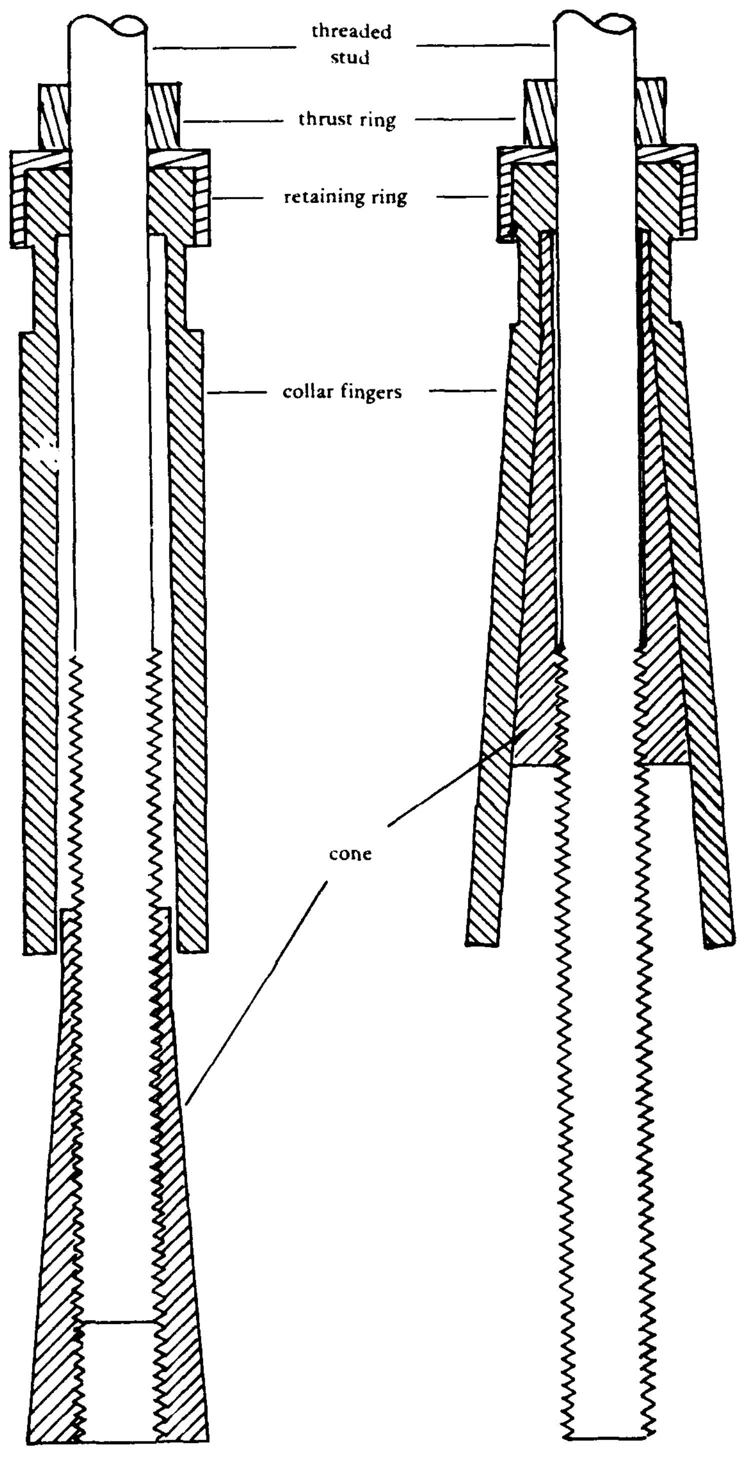


Figure 4-42. Prototype rockbolt before and after expansion of the collar.



(a) Ready for installation.   (b) Collar expanded.

Figure 4-43. Cross section of prototype rockbolt.

The cable is immobilized by installing the rockbolt in pairs at intervals along the cable. Spacing between the rockbolt pairs will be established during design of the installation and will depend on rock type and hardness, expected wave loading, and size of the cable. Installing the bolts in pairs with a saddle spanning the cable (Figure 4-44) assures a positive connection between the cable and the seafloor, reduces bending stresses in the rockbolt, and minimizes lateral movement of the rockbolt in the hole.

Single rockbolt clamps have been used in the past for securing small cables, but this is only recommended when the cable diameter is approximately equal to or smaller than the shank of the rockbolt or when cable installation is in deep water where wave loading is expected to be minimal. Care must be taken in the design of single bolt clamps to assure that the rockbolt can be properly tightened (Figure 4-45).

When rockbolts are used in conjunction with split-pipe, the pipe itself can be used as the clamp. The holes in the flange are used as a drill jig and the rockbolts are installed through the flange into the predrilled hole. This technique limits the selection of the rockbolts to 5/8-in.-diam masonry stud bolt and is therefore only suitable where hard competent rock seafloors exist. To provide adequate penetration of the anchor into the rock a minimum length of 12 in. is required. If zinc anodes are to be utilized for cathodic protection, rockbolts up to 20 in. long are required (Figure 4-46). Since the use of rockbolts reduces the number of fasteners holding the split-pipe together, two of the diagonally opposing center four holes in the pipe flange should be used in order to minimize the effect on the integrity of the split-pipe assembly. If rockbolts are to be used with split-pipe on weak or porous seafloor materials, then a bolt larger than 5/8-in.-diam will be required along with a saddle which spans the split-pipe (Figure 4-47).

The torque required to properly set the anchor portion of the rockbolt varies with both the size and type of bolt used and hardness of rock. In general, however, it has been found that a minimum torque of 40 ft-lb is required to assure proper expansion of the masonry stud bolt, while 75 to 100 ft-lb is required for the mine tunnel bolt. The torque applied when securing the rockbolt to the clamp or split-pipe will usually differ from the setting torque.

Very few sites have a seafloor composed of homogeneous, competent rock without any sand cover; therefore, the selection of points on the seafloor for installation of rockbolts is one of the most crucial phases of the operation. In most cases, the location of the rockbolt installations will not coincide exactly with the design spacing because of the presence of sand pockets, loose boulders, or fractured rock.

Prior to the installation of any rockbolt immobilization, the cognizant on-site engineer, who has knowledge of the immobilization design requirements, should inspect the cable systems, and identify and mark the location of rockbolt installation sites. If these sites differ significantly from the design spacing, the actual spacing should be determined and calculations made to determine if the installation will be adequate.

Figure 4-44. Cable clamp and rockbolt immobilization configuration.

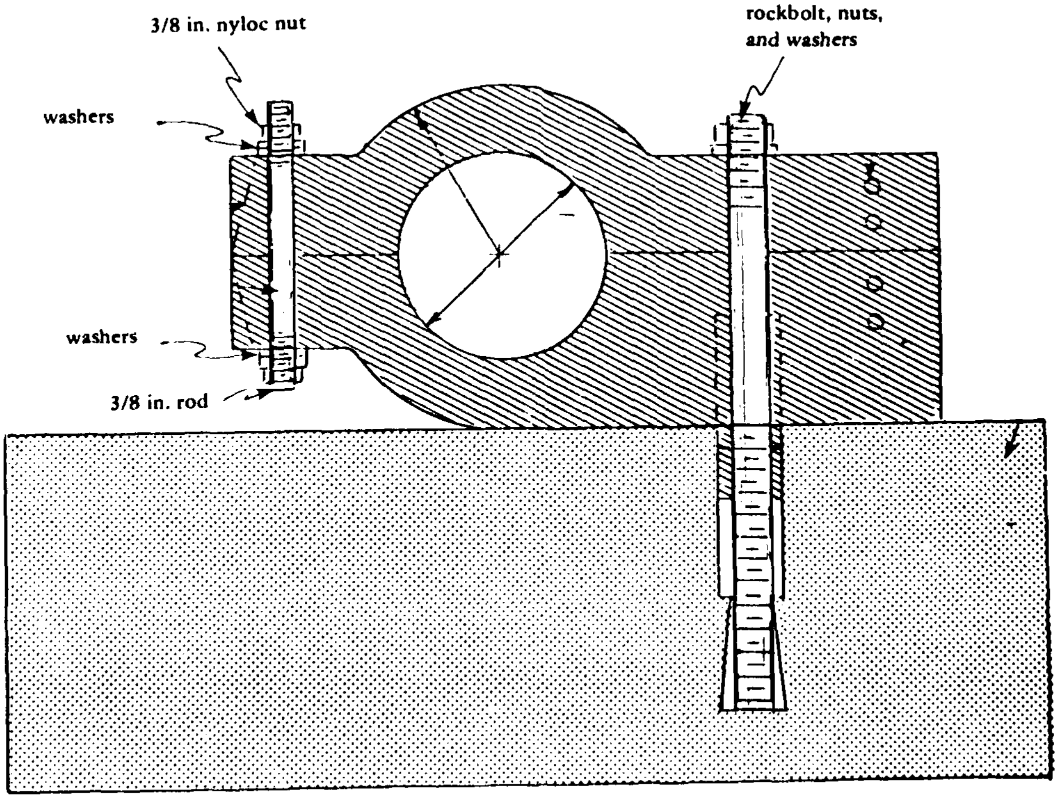


Figure 4-45. One-bolt cable clamp assembly.

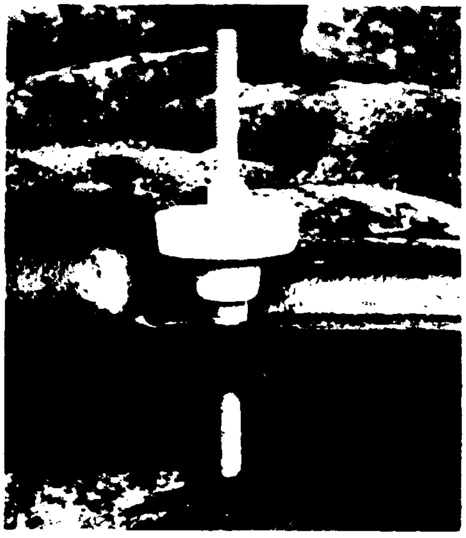


Figure 4-46. Cathodic protection for rockbolt immobilization system.

Premarking of the installation site saves a great deal of time and eliminates confusion; it allows the construction diver to concentrate on rockbolt installation and moving of equipment along the seafloor without having to stop every few feet to investigate the quality of the seafloor rock.

Design Considerations. A considerable amount of data on the holding capacity of various types of rock samples has been obtained and is presented in Figures 4-48 through 4-50 and Table 4-13 (Brackett and Parisi, 1975).

A wide scattering of the data points appears in these figures. Because of the number of tests performed, several different rocks of the same material and compressive strength had to be used. However, internal voids and veins of harder or softer material as well as cracks and faults were noted in many cases. These factors have an obvious effect on failure load as does size of the rocks - smaller rocks tend to split when the bolt is loaded.

The general trend apparent in the data is an increase of failure load with bolt diameter and embedment depth. A comparison of results for underwater and surface tests indicates that underwater failure loads tend to be smaller. However, it was necessary to use smaller rocks in the underwater tests, and on several occasions the rocks split at loads smaller than those for other modes of failure.

Observations made during the pullout tests indicate that the mode of failure is dependent on the type of rock, depth of embedment, and diameter of the bolt. Tests to determine the correlation between initial torque and failure load revealed that a minimum of about 40 ft-lb is required to properly set the anchor; increasing the torque above this minimum value has no effect on the ultimate holding strength of the anchor.

The results of long-term testing (Tables 4-14 and 4-15) revealed that the bolts installed in soft sandstone showed a decrease in holding strength of about 33% after 6 months, but no further loss in strength was noted after 15 months of exposure. Those installed in the basalt showed a 15% decrease in holding strength after 6 months, while the strength of the bolts installed in the cementitious siltstone did not decrease during 15 months of exposure.

No measurable loss of material was noted, although some corrosion products were evident; no correlation could be found between long-term failure load and the amount of corrosion. Because sandstone is very soft, it is possible that, over a long period of time, the pretensioning load caused creep of the rock behind the wedge, allowing it to expand slightly.

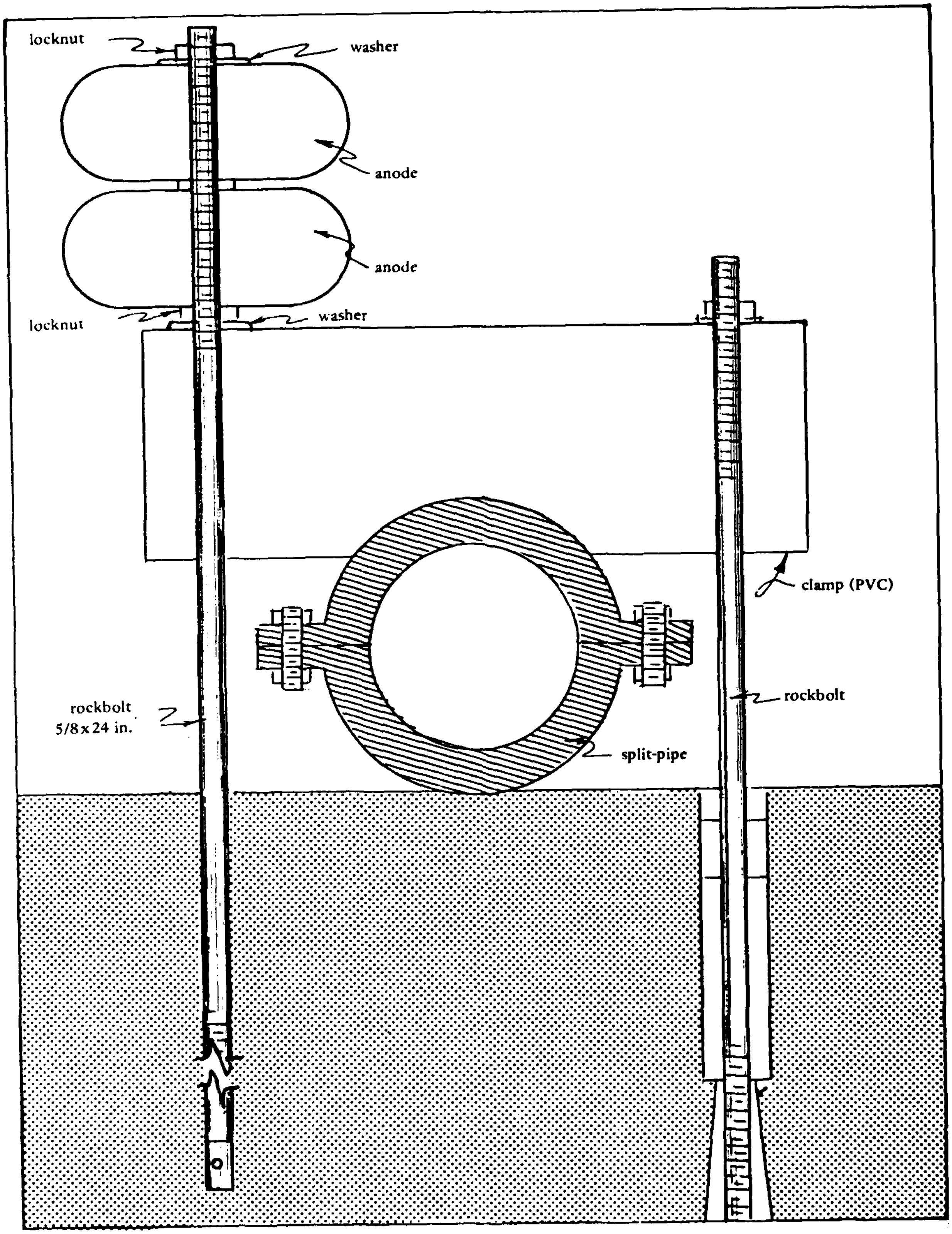


Figure 4-47. Split-pipe installation.

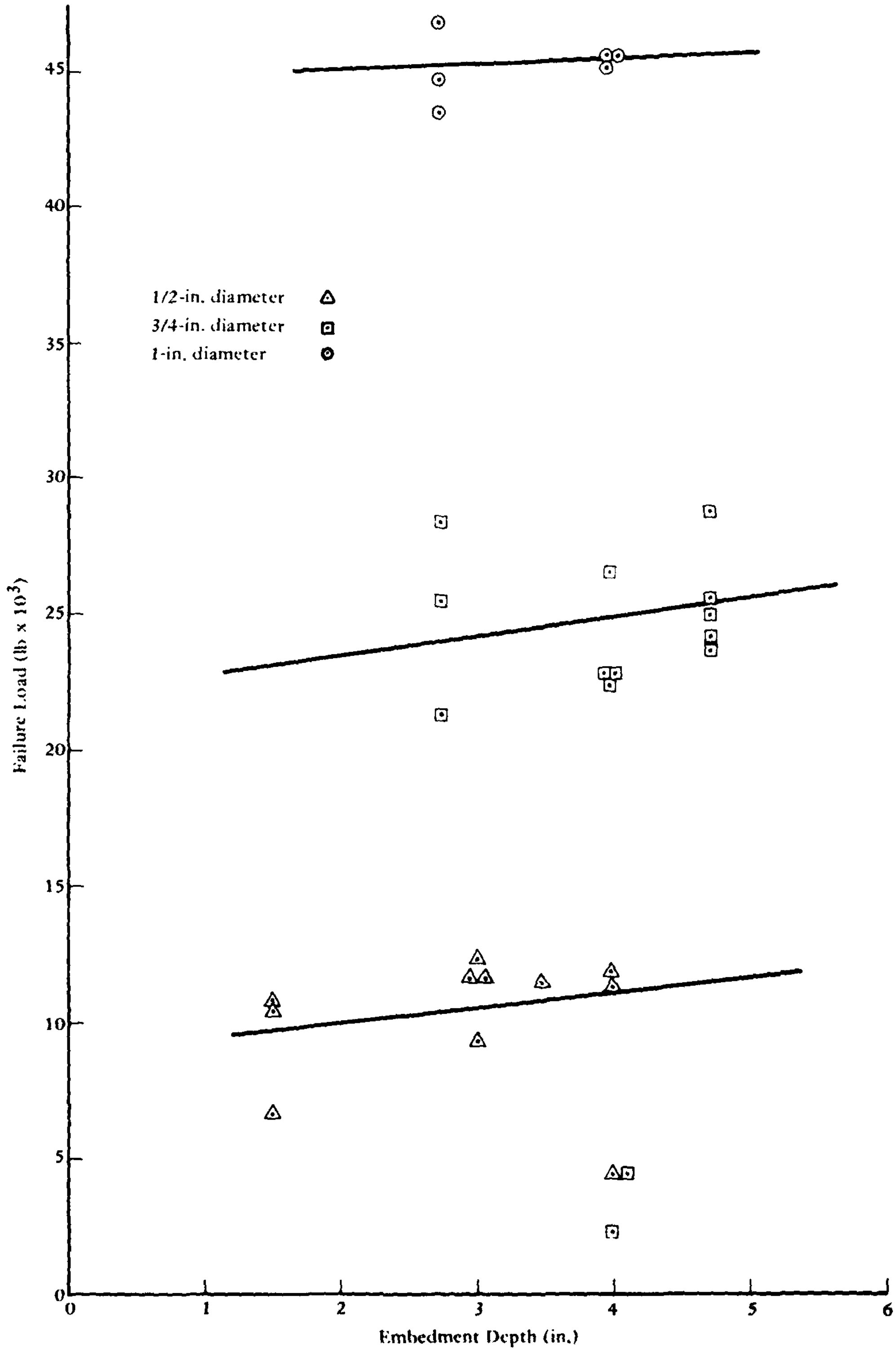


Figure 4-48. Failure loads for Red Head bolts embedded in granite (12,000 psi).

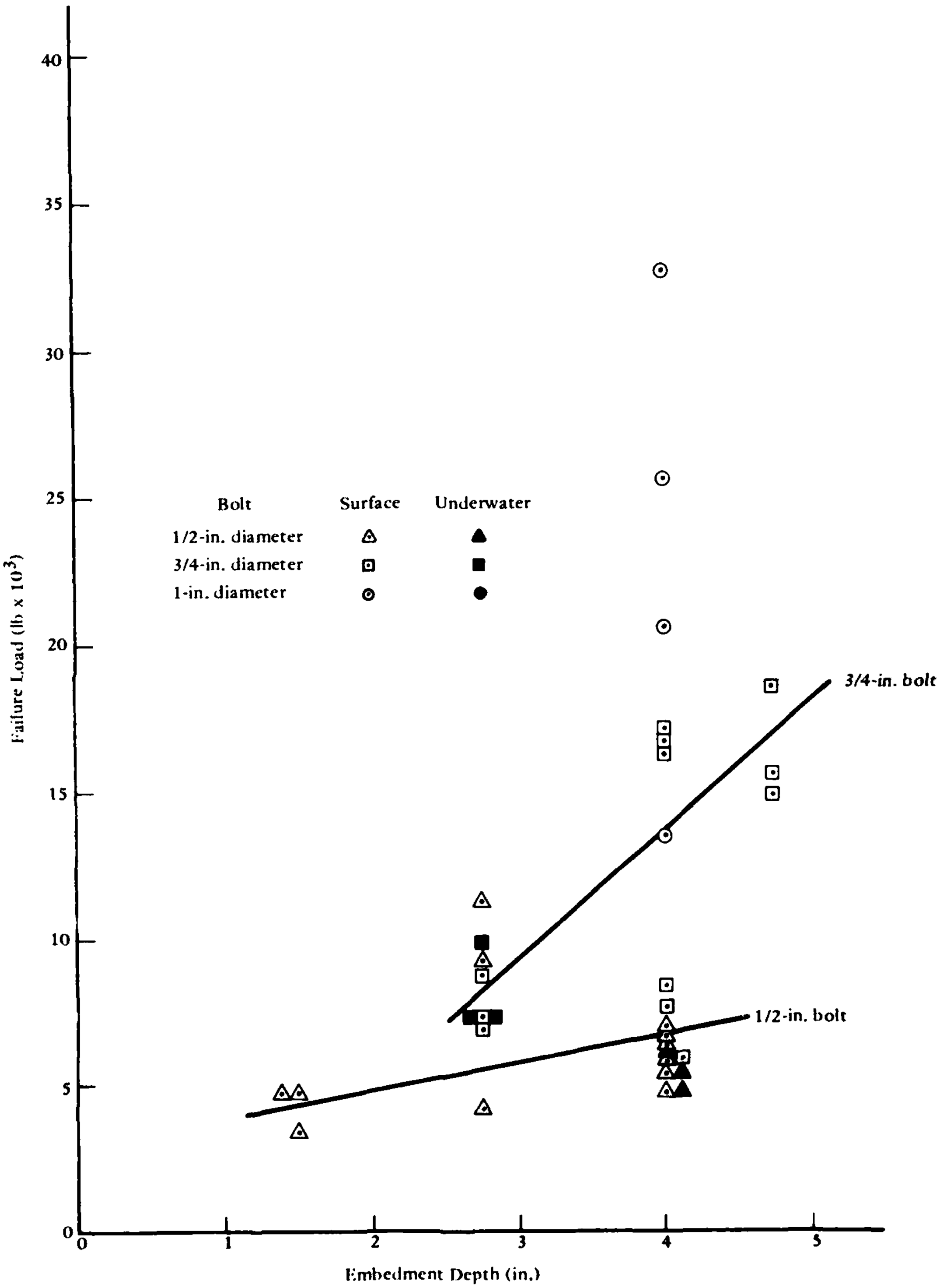


Figure 4-49. Failure loads for Red Head bolts embedded in sandstone.

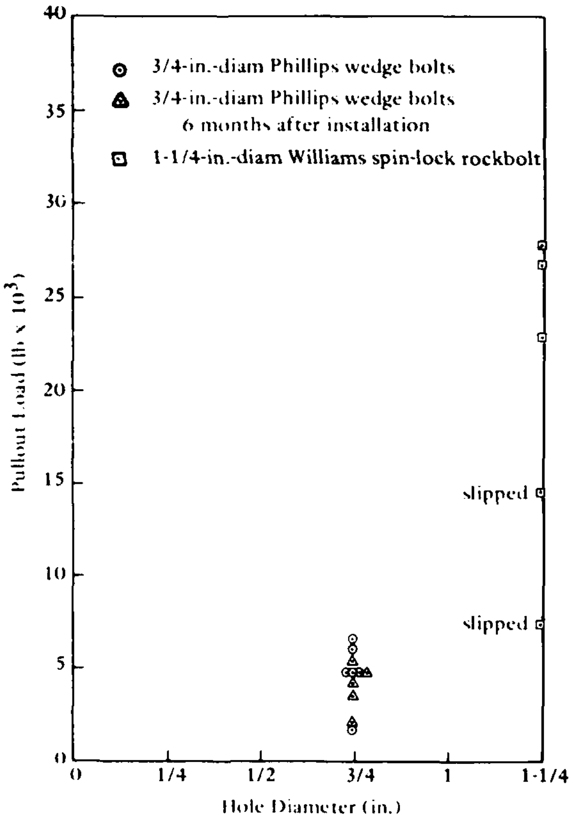


Figure 4-50. Failure loads for various types of rockbolts in vesicular basalt.

To determine what correlation, if any, exists between compressive strength of the rock and failure load, those data were plotted (see Figure 4-51). With only one exception, compressive strength seems to give a good indication of failure load. The vesicular basalt tested had many voids in it; this factor probably accounts for the low failure loads obtained.

Because of the variation of rock type and structure at various sites and the wide scatter of data presented in the previous graphs it is recommended that preliminary tests be conducted on-site with care being taken to select rocks similar to those where the actual installation will take place. Underwater tests are not necessary. If actual holding capacity tests

Table 4-13. Test Data for Various Rockbolts Embedded in Coral (Kauai, Hawaii)

Type of Rockbolt	Depth (ft)	Bolt Diameter (in.)	Load (lb)	Mean Load (lb)	Standard Deviation (lb)
Masonry Stud	surfzone surfzone surfzone	3/4 3/4 3/4	4,635 7,130 4,990	5,585	1,350
	60 60	3/4 3/4	0 0	–	–
Mine Tunnel	60 60 60 60 60	1-1/4 1-1/4 1-1/4 1-1/4 1-1/4	1,780 2,850 2,140 1,780 4,635	2,637	1,200
Coral	60 60	1-1/4 1-1/4	6,000 12,000	–	–

are not conducted the site survey should at least include measurement of the compressive strength of the rock, which can be used as an indication of expected holding strength, if the physical structure of the rock (i.e., presence of voids, etc.) is investigated.

Table 4-16 summarizes the parameters that affect the holding strength of seafloor fasteners.

Table 4-14. Long-Term Test Data for 3/4-In.-Diam Phillips Wedge Bolts Embedded 4 Inches at San Nicolas Island

Sample No.	Failure Load (lb)		
	Immediately After Installation	After 6 Months	After 15 Months
Soft Sandstone (compressive strength ≈ 600 psi)			
1	6,773	2,852	1,568[a]
2	5,347	2,495	3,707
3	4,278	2,139	3,707
4	4,278	3,921	3,422
5	6,417	3,921	–
6	713[a]	4,278	–
7	–	4,456	–
8		3,921	–
9	–	3,921	
10	–	4,634	–
11		2,852	–
Mean Value	5,418	3,580	3,612
Std Dev	1,166	845	164
Cementitious Siltstone (compressive strength ≈ 15,000 psi)			
1	24,240	29,230	26,380
2	27,800	29,230	30,660
3	26,380	–	32,800
4	27,810	–	22,820
5	29,090	–	–
6	29,230	–	–
Mean Value	27,090	29,230	28,160
Std Dev	1,687	–	4,452

[a] This number excluded from mean value and standard deviation calculations.

Tensioning of the rock bolt against the clamp or split-pipe improves immobilization by increasing the friction force between the cable (split-pipe) and the seafloor (see Section 6.3.2).

Although coefficients of friction may vary widely, the pretensioning force may be estimated from the torque produced when tightening the nut from the equation:

Table 4-15. Long-Term Test Data for 3/4-In.-Diam Phillips Wedge Bolts Embedded 4 Inches at Anacapa Island

[All tests performed in basalt rock.]

Site No.	Failure Load (lb)	
	Immediately After Installation	After 6 Months
1	4,800	5,500
	6,000	3,400
2	1,700	2,000
	4,800	0
3	4,800	4,500
	6,500	4,800
Mean Value	4,766	4,040

$$T_B = \frac{2N\,T_N}{d_B\left\{\left[\dfrac{\sin\alpha_t + \mu_o(\cos\alpha_t/\cos\beta_t)}{\cos\alpha_t - \mu_o(\sin\alpha_t/\cos\beta_t)}\right] + \mu_N\left(\dfrac{d_N}{d_B}\right)\right\}} \qquad (4\text{-}3)$$

where T_B = pretension load on rockbolts (lb)

N = number of rockbolts/clamp

T_N = torque on each bolt (lb-in.)

α_t = lead angle of thread

β_t = one-half included thread angle

μ_o = coefficient of thread friction

μ_N = coefficient of friction between nut and washer

d_B = bolt diameter

d_N = mean diameter of washer face on the nut

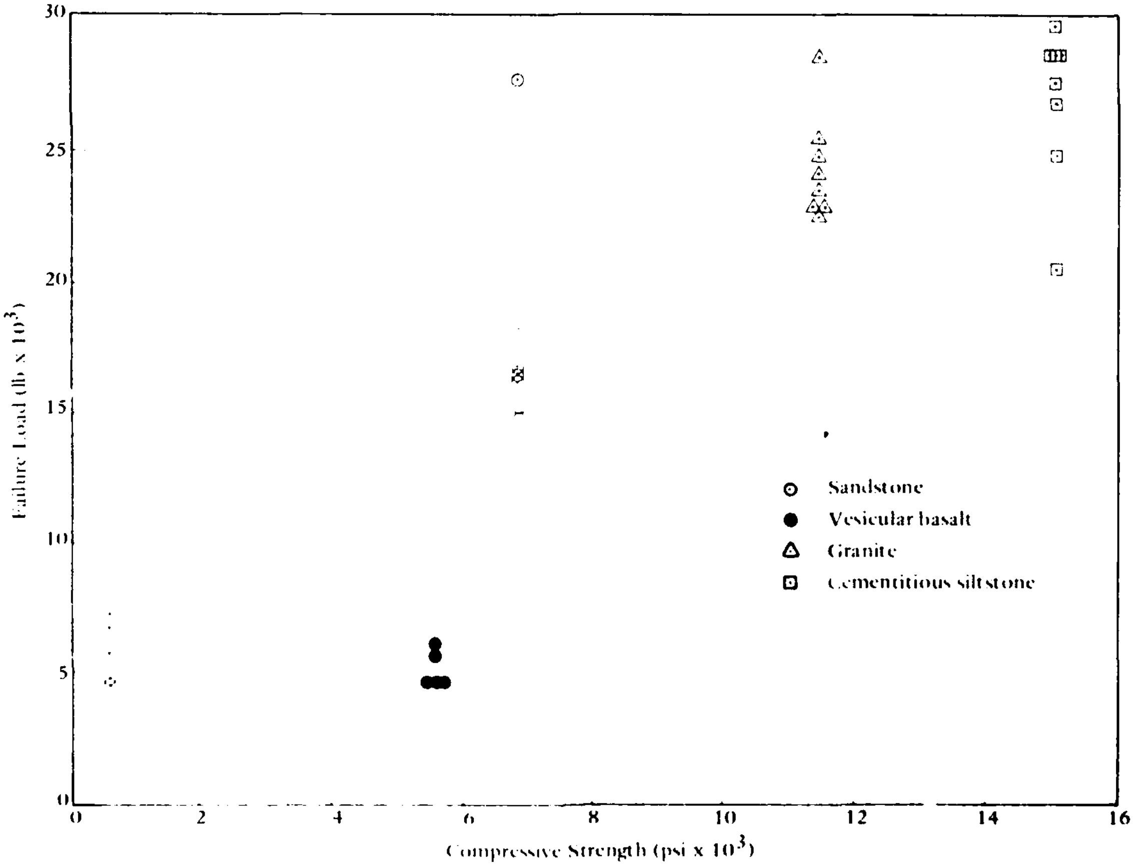


Figure 4-51. Failure load for 3/4-in.-diam wedge anchors embedded 6 inches in different types of rocks.

Table 4-16. Parameters Affecting Holding Strength of Seafloor Fasteners

Parameter	Effect on Holding Strength	Comments
Bolt diameter	The bolt diameter determines the ultimate potential holding strength possible for a given size bolt and the ultimate tensile strength	If all bolts have the same ultimate tensile strength, the failure load of the bolt will vary as the square of the diameter.
Anchor configuration		
Length and diameter of collar	The length and diameter of the anchor collar affect the stress produced in the seafloor rock. An increase in size of the anchor collar will decrease the stresses in the rock, thus reducing the chance of failure due to localized crushing or splitting of the rock.	An increase in anchor diameter requires an increase in drilling time. The trade-off between installing one large rock bolt or several small bolts in a padeye configuration should be considered.
Type of collar	A one-piece split collar has proven to give slightly higher pullout loads than the two-piece collar design for the same size fastener.	
Embedment depth	An increase in embedment depth produces almost a linear increase in holding strength up to the point where either localized crushing of the rock occurs around the collar or the ultimate tensile strength of the bolt is exceeded.	As a general rule a 6-inch embedment is sufficient to eliminate failure due to surface fracturing of the rock. Bolt diameter, competency of the rock, and presence of hard or soft substrata should be considered before determining the minimum embedment depth.
Duration of installation	There is not sufficient data at the present time to predict the exact effect of corrosion on the long-term holding strength of the fasteners tested. A trend toward a slightly reduced holding strength was detected after as little as 6 months of exposure.	The use of zinc anodes along with periodic inspection and replacement of spent anodes should ensure the integrity of the fastener for many years.
Initial torque	Initial torques of 40 ft-lb for the masonry stud anchor and 100 ft-lb for the spin-lock rock bolt were found to be necessary to properly set the anchor. Torquing the bolts above these values have no effect on the holding strength of the bolt.	The masonry stud anchors could be properly set by a diver using a hand wrench, but the use of an hydraulic impact wrench is recommended to ensure proper setting of the spin-lock rock bolt.
Compressive strength of rock	The holding strength of a given size fastener is almost linearly dependent on the unconfined compressive strength of the rock.	The presence of internal voids or fractures in the rock must be investigated before using compressive strength as a design criterion.
Installation of fasteners on land versus underwater	There appears to be a slight decrease in holding strength for bolts installed underwater compared to the same installation on land. The wide scatter of data points makes it difficult to quantitatively determine the magnitude of this decrease in failure load. However, if a normal safety factor is applied to the results of land tests, a realistic safe working load for the underwater installation should be obtained.	Care must be taken when using land tests to predict underwater performance. The test installations must be conducted in rock representative of that actually found at the seafloor work site. This analysis should include: size, porosity, presence of voids and fractures, presence of biological organisms, such as those in coral, that may have a significant effect of the holding strength of the fastener.

For semifinished hex nuts $d_N/d_B = 1.25$, and for average nuts and bolts a value of 0.15 can be used for μ_o and μ_N. These values may vary, depending on thread finish, accuracy, and degree of lubrication, but are generally in the range 0.12 to 0.20 (Shigley, 1963). The thread lead angle varies with pitch and diameter of the bolt. It can be calculated from the equation

$$\alpha_t = \tan^{-1}\left(\frac{1}{n_t \pi d_B}\right) \tag{4-4}$$

where n_t = number of threads/in.

d_B = bolt diameter (in.)

For American standard screw threads β_t is equal to 30 degrees.

Because of the scatter of holding capacity data, rockbolt immobilization systems should not be designed with a factor-of-safety <4.

Since pretensioning of the rock bolts produces a compressive stress field in the seafloor rock, the spacing of the rock bolts in the clamp is critical. Table 4-17 provides the recommended minimum spacing between rockbolts to achieve 100% and 80% of the maximum holding capacity of each rockbolt. Placing rockbolts closer together than the minimum for 80% capacity is not recommended.

Table 4-17. Minimum Recommended Rockbolt Spacing

Bolt Diameter (in.)	Holding Capacity (in.)	
	100%	80%
1/4	3	1-1/2
5/16	3-1/4	1-5/8
3/8	4	2
1/2	5	2-1/2
5/8	6	3
3/4	7	3-1/2
7/8	8	4

Support Requirements.

MANPOWER.* The manpower requirements for the installation of rockbolts will depend on the length of cable, rockbolt spacing, weather window, and water depth. This type of installation has been accomplished with as few as six diving-qualified personnel when the work was done in calm shallow water where the operation could be supported from shore. A larger scale project in deeper water (40 to 60 feet) utilized a crew of 29 personnel with a minimum of 24 divers being used each day.

*The information provided in this section is based on limited data from a few previous installations and as such is intended only as a guide in developing preliminary cost estimates. Since environmental conditions vary considerably from site to site the final decision on the number of personnel required to safely conduct the operation must be left to the discretion of the diving officer.

EQUIPMENT. The type of equipment required for rockbolt installation will vary slightly, depending on the type and number of rockbolts to be installed. Major equipment includes a rock drill and power source, cable clamps (or split-pipe), and rockbolts. The following table lists the minimum equipment required to support the immobilization operation.

Two types of hydraulic-powered rock drills have been developed for installation of rockbolts. The hand-held drill (Figure 4-52) is capable of drilling holes from 1/4-in.-diam to 1-1/2-in.-diam and depths of 18 in. The heavy-duty rock drill (Figure 4-53) produces holes between 1-1/2 and 4 in. in diameter to a depth of 4 ft. Pneumatic rock drills have been used in the past for installation or rockbolts but are not recommended because of the dangerous percussion produced by the exhaust gas. The power source utilized with the hydraulic rock drills should be capable of flow rates of 10 gpm and pressures up to 2,000 psi.

Equipment	Requirements
Diving Gear	as required
Rock Drill:	
Model #74-6-0275 (for 1/4" to 1-1/2" ϕ holes) or	2
Model #75-9-0475 (for 1-1/2" to 4" ϕ holes)	2
Drill Maintenance and Repair Kit	1
Hydraulic Power Source (10 gpm, 2,000 psi)	1
Hammer (3 lb with short handle)	1
Wrench (rachet or impact)	2
Hydraulic Hose (3/4-in. pressure and return, 250 ft)	1
Cable Clamps	as required
Rockbolts	as required
Surveyor's Tape	as required
Carbide-Tip Rock Drill Percussion Bits (diam determined by size of rockbolt)	10
Underwater Tool Bag	1
Drilling Jig	as required
Surface Support Platform	1
Hydraulic Oil (spare)	55 gal
Mooring System	as required

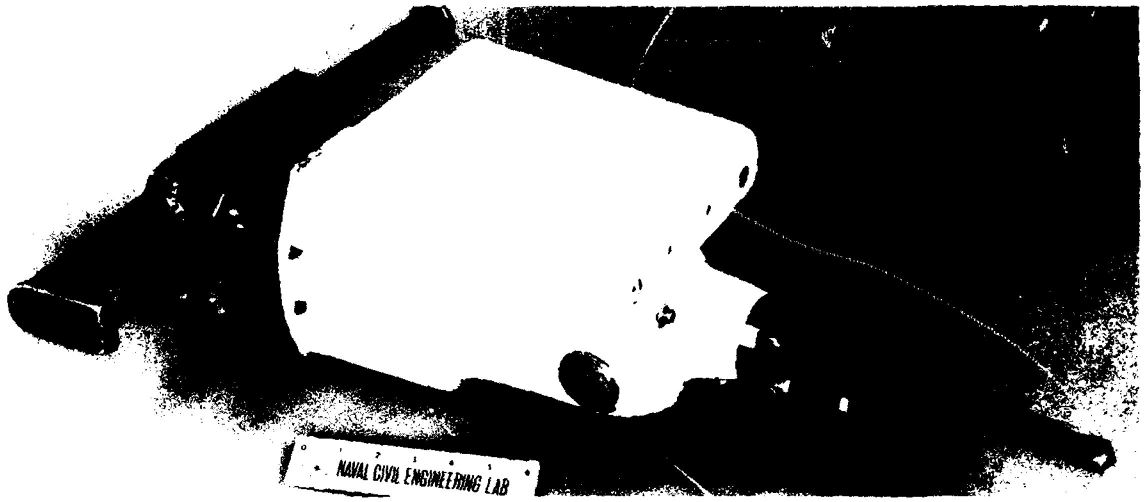


Figure 4-52. Hand-held hydraulic rock drill.

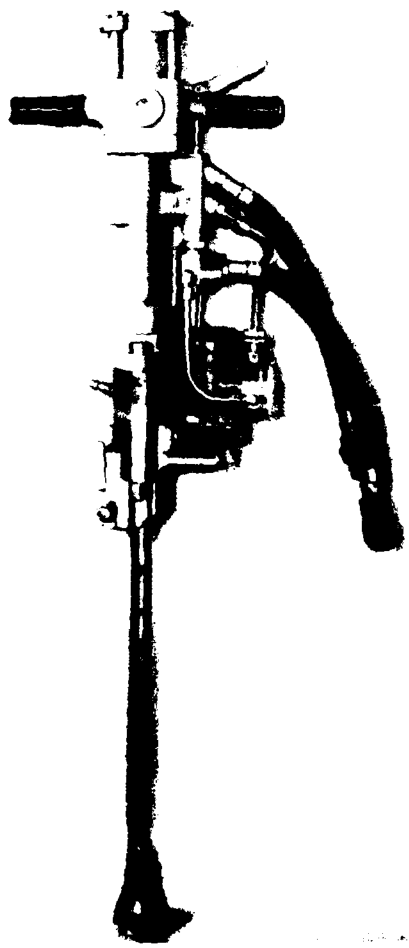


Figure 4-53. Heavy-duty hydraulic rock drill.

Installation Time Estimates. The installation times for rockbolts depend on (1) the diameter and depth of the hole; (2) the type of rock being drilled; (3) the spacing of the bolts, which may involve moving the drill considerable distances; (4) the actual installation of the fastener; and (5) the time required to change diving teams. An indication of the required drilling time can be obtained from Figures 4-54 and 4-55. For example, a hole for installing a 3/4-in.-diam by 6-in.-long fastener in granite would take approximately 1-1/2 minutes to drill. The time required to move the drill to the next installation site must be evaluated on a project-to-project basis, and it will depend on distance between installations, type of diving gear used, type of surface support, and type of bottom over which the divers must move. Installation of the bolt itself varies from about 3 minutes for a masonry stud bolt to about 5 minutes for a mine tunnel or coral bolt.

Table 4-18 lists the actual times required for previous rockbolt installation operations.

Selection Factors.

BOTTOM MATERIAL AND TOPOGRAPHY. Rockbolt immobilization is practical only on exposed rock and coral seafloor. Sand cover, even as thin as 2 in., makes rockbolting very difficult and time consuming. Broken rock and small boulders at the exact locations specified in the stabilization system design will affect the installation of rockbolts. Rough or irregular topography will not have an appreciable effect on the installation of the rockbolts, but it will require slightly longer times for the installation because of increased problems in moving equipment from one site to another along the seafloor.

WAVES. To apply rockbolts in the surfzone, the surf must be less than 1 ft. Sometimes the requirement can be circumvented by taking advantage of large tidal variations to eliminate working in the actual surfzone. Swells greater than 6 ft will pose problems in water depths between 20 and 60 ft.

CURRENT. Because the diver must be in a vertical position when drilling the holes for rockbolt installation, currents or surge greater than 1 knot present problems. At best it will make the installation more time-consuming but usually results in bent or broken drill bits.

LOGISTICS. Little if any specialized logistics support is required for this type of operation. Previous operations have been successfully completed using either a LARC V or LCM6 for surface support. The number of divers utilized will depend on the weather window and manpower available.

WEATHER WINDOW. Weather conditions must be favorable enough to allow the diving boat to stay on-station for long periods of time. For any significant work to be accomplished a minimum of 4 hours on-station without interruption of operation is required. With ideal conditions, approximately 400 ft of cable can be stabilized with rockbolts by each diving station in a 10-hour workday.

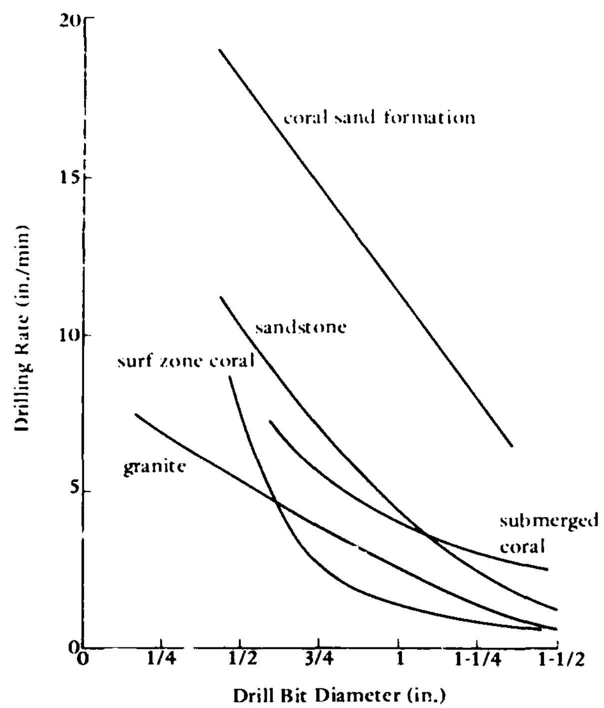


Figure 4-54. Drilling rate for hand-held rock drill in various seafloor materials.

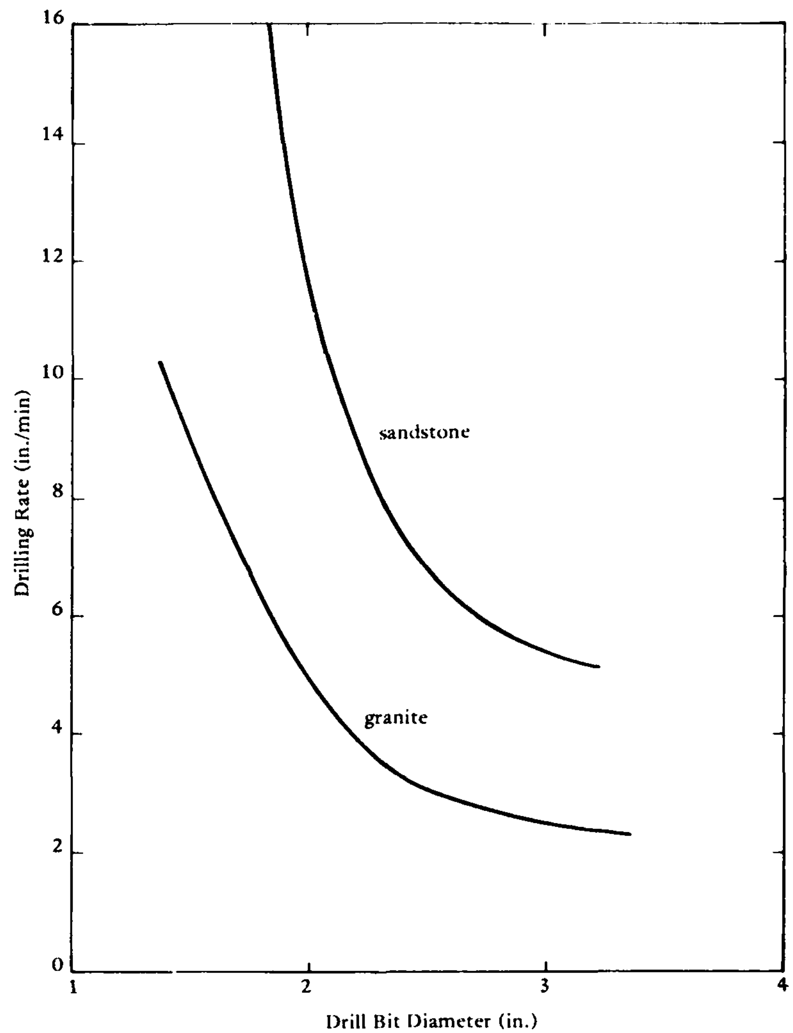


Figure 4-55. Drilling rate for large-hole rock drill in various seafloor materials.

Table 4-18. Rockbolt Installation Times

Type of Rockbolt	Number Installed	Length of Cable (ft)	Diving Time (manhr)	Average Installation Time Per Bolt (hr)	Total (hr)	Days to Complete
Coral Bolt						
1-1/4-in. diam x 18 in. L	183	7,000	83.24	0.29	52.75	8
Masonry Stud Bolt						
1. 5/8-in. diam x 12 in. L	64	500	20	0.2	12.8	13
2. 5/8-in. diam x 22 in. L	60	400	15	0.2	12	3
3. 5/8-in. diam x 12 in. L	26	300	9.7	0.19	4.85	1

VISIBILITY. Visibility less than about 3 ft will slow the installation process considerably, especially if it is associated with current or surge conditions.

HAZARDS. Rockbolts greatly improve a cable's ability to withstand the hydrodynamic forces from waves and current. The ability to withstand anchor drag will depend upon accurately predicting the magnitude of these forces at the particular site and designing the system to withstand them. Rockbolt immobilization provides little if any protection against ice scoring and trawler drags; however, the latter is usually not associated with rocky seafloor areas.

WIND. Winds greater than 20 knots make diving operations difficult, especially if the wind is acting over a significant fetch.

DESIGN LIFE. With proper selection of materials and identification or realistic design loads, rockbolt immobilization systems can be designed and installed to meet virtually any design life requirement.

LENGTH OF PROTECTED CABLE. The length of protected cable has very little impact on the feasibility of rockbolting; however, for long cable immobilization operations, sufficient personnel must be available to assure its completion within the weather window.

4.4 BURIAL

Burial provides protection by allowing the cable to be placed below the surface of the seafloor. The effectiveness of these techniques depends on their ability to remove the cable from the environment where conditions which are hazardous to the cable may exist. Since burial eliminates the influence of the environmental hazards rather than providing a means to resist them, the design theories presented in Chapter 6 are not applicable. The selection and implementation of one of these techniques depends, therefore, on economics and the ability of the available equipment to bury the cable to the required depth (to avoid the potential hazard).

The techniques discussed in this section include: (1) self-burial, (2) jetting, (3) dredging, (4) explosive excavation, (5) mechanical trenching, and (6) drilled hole.

4.4.1 Self-Burial

Background and Description. Most armored nearshore cables lying on sands, silts, and soft clays in the nearshore zone will sink below the sediment surface because of the high unit weight of the cable and the strength reduction and scour of the underlying seafloor sediments caused by waves and currents. The sediment strength reduction on sand is caused by pore water flow upward and out of the seafloor, induced by the varying pressure field of passing waves. Self-burial or natural burial may occur rapidly, removing a cable from view the same day it was placed.

The natural burial mechanism in nearshore sands and silts requires that the object to be buried have a bulk density of at least 119 lb/ft^3 (Van Daalen and Van Steveninck, 1970). The bulk densities of typical single- and double-armored nearshore communications cables range from 194 to 280 lb/ft^3 indicating that armored cable varieties will self-bury 1 to 3 feet in sands and silts due to wave action (Valent and Brackett, 1976).

On soft cohesive soils, such as clays and clayey silts, which may be encountered in areas of high sedimentation (e.g., on some river deltas), heavy-armored cables will often sink from their own weight, irrespective of wave and current forces. Here, the very soft clays are not strong enough to support the cable, and it sinks due to a bearing capacity failure until reaching an elevation where the soil strength is sufficient to support it. Though pipelines on such materials sink considerable distances, causing pipeline damage through distortions at wellhead fittings (Ledford, 1953), damage to a cable system due to this mechanism is unlikely.

Protection of electrical cable systems from sand abrasion can be economically achieved by relying on waves and currents to self-bury the cable. This technique is entirely satisfactory provided the following favorable conditions exist:

(1) Sufficient slack can be provided to permit the cable system to "follow" the seafloor surface during periods of local nearshore erosion.

(2) The cable route does not pass over rock that will be exposed at some time during the life of the cable system. Exposure of the cable on the rock outcrop to wave and current forces may subject that cable to excessive intermittent abrasion and motion.

(3) Little probability of damage due to dragging anchors exists.

Procedures. The only procedure involved, other than making sure during the design stage that the self-burial concept will work in this cable route environment, is to place the cable on the seafloor along the desired cable route and to allow sufficient slack to ensure that the cable will not be restrained from following the seafloor profile.

Support Requirements. There are no support requirements beyond those required for the cable laying itself.

Selection Factors.

BOTTOM MATERIAL AND TOPOGRAPHY. The nature of the seafloor material; the thickness of the noncohesive sandy seafloor layer; the topography of the underlying hard, dense layers; and the potential for cable damage on the hard layers, if exposed, all play an important part in any decision to select a cable route and to rely on self-burial. Cables should be expected to be buried by waves and currents only in noncohesive materials. The existence of clay layers, or gravel layers, or cemented layers, or rock erratics will prevent further downward migration of the cable ahead of seafloor erosion; thus, self-burial cannot be relied on to provide necessary protection where such materials may be encountered.

WAVES AND CURRENTS. Waves and currents are significant to a decision to rely on self-burial only when establishing how much slack should be left in the cable to accommodate changes in bottom elevation (i.e., when predicting local scour).

HAZARDS. The potential for hazard occurrence - in particular, damage due to dragging anchors, ice scoring, and earth-mass movements - must be thoroughly evaluated when considering natural burial as cable protection. Natural burial can be counted on for only a few feet of cover at most, certainly nowhere sufficient to protect a cable from being engaged by a dragging anchor or the keel of most ice masses (Valent and Brackett, 1976). Similarly any downslope movement of seafloor masses, whether slow creep movements, as observed down some canyons (Dill, 1964), or rapid flows of fluid soil (Terzaghi, 1956), will likely include, and often will damage, electrical cables in their path.

4.4.2 Jetting

Background and Description. Cable burial by jetting can be accomplished through two quite different mechanisms. The first involves using large jets to erode and displace seafloor material, leaving an open trench into which the cable is inserted. This technique is workable in most noncohesive materials (except for those with large gravel not movable by the jets) and in many cohesive soils (except for those too highly consolidated to erode with low pressure jets) (e.g., Armstrong, 1975; Anon., 1972). A variation of this concept of trench excavation by water jet is that of diverting the slipstream of a ship's screw to create a giant, low pressure jet to scour a very wide trench in the shallow nearshore seafloor (Klopfenstein, 1974).

In noncohesive soils, excavation of the open trench requires displacement of large volumes of soil because of the flat angle of repose of the side slopes. Thus, jetting with a few large jets to excavate a trench is not generally attractive in such materials. When in sands, free of gravel and cohesive soil layers, fluidization is the preferred cable burial technique, because it consumes far less energy, especially for deep burial. In this second cable burial technique, only a narrow slit of fluidized soil is formed during the passing of a plow-like stinger and electrical cable guide chute. Fluidization is accomplished through the many small jets in the leading edge of the stinger which erode the sand in front of the stinger and suspend that sand in the upward flowing jetting water. If the jetting stinger does encounter cohesive soils, then significantly higher drawbar pull augmented by vibration of the stinger may be necessary to continue burial (Welte, 1972).

Procedure. Two procedures may be used to jet cables into the seafloor. First, the cable is laid on the seafloor along the planned route, and then the jetted "trench" is excavated beneath or beside the cable and the cable settles or is placed in the bottom of the trench (Armstrong, 1975). Second, the cable is fed to the jetting machine as it advances along the cable route. The cable may be stored on a surface platform, probably the same platform powering the jetting device (Welte, 1972), or, for small diameter, lightly armored cables, the cable may be fed from a reel mounted on the jetting device itself (Anderson, 1974). When the cable is fed to the jetting machine, it is commonly run through a guide chute directly to the bottom of the jetted slot.

Support Requirements and Installation Time. Jetting in the nearshore zone for electrical cable burial is most commonly accomplished by supplying water from the surface support platform at high volume. The jetting nozzle is carried on a bottom-resting sled or vehicle. The smaller jetting systems, capable of 1- to 6-ft depth of burial per pass, often consist of the seafloor nozzle or stinger equipped vehicle and its own specially designed power supply. Jetting water flow rate is at least 500 gal/min and the pressure no more than 150 psi. These can be

operated from a work boat of opportunity (Blake and McBride, 1972). Depending on the depth of burial required and on the nature of the seafloor materials, the burial rate will vary from 200 to 1500 ft/hr.

In those areas where waves and swells can be counted on to be insignificant, the jetting system can be mounted on a barge rather than on a seafloor traversing vehicle. Embedment of cables can then approach 30 feet below the seafloor (Welte, 1972). However, again, such barge-mounted, plow-like stingers are not amenable to use in the usual sand nearshore zone. They are mentioned here because they are especially useful when burying cables crossing channels which are likely to be dredged or to experience anchor drag incidents.

Selection Factors.

BOTTOM MATERIAL AND TOPOGRAPHY. The nature of the seafloor material and the thickness of the respective layers will determine whether jetting for cable burial is feasible by a given technique and will determine whether the advance rate is sufficient to economically justify burial by jetting. Any examination of the bottom material must be very thorough and requires deep cores tied together by an acoustic survey. A geologist experienced in coastal environments is often indispensable to the site survey and evaluation team. Above all, care should be taken in properly identifying and assessing the extent of clay and gravel layers and transported rock.

Such undetected or unappreciated layers and inclusions can be responsible for the failure of an installation effort. For example, the small, towed, jetting sled developed for work in the Arctic (Anderson, 1974) failed to perform as designed because of large blocks of weathered rock in the upper 5 feet of the seafloor profile. The stinger was forced to ride up and over these blocks leaving the cable with too little cover in many places. A similar problem developed during burial of a pipeline crossing the Menai-Straits, England (Davis, 1974). Cohesive layers were encountered in the tidal areas which were not amenable to fluidization, and other excavation techniques were required to complete the trenching in order to lower the pipe to the required grade.

Topography is not often a consideration in applying jetting techniques because normally material which will exist in landforms sufficient to impede a jetting device will be composed of material not suitable for jetting anyway.

WAVES AND CURRENTS. Waves and currents can have very great impact on the success of a cable burial project by jetting. Waves apply significant overturning forces to the jetting vehicle and cause reduced seafloor bearing capacity, resulting in sinkage of vehicle skids, wheels and tracks. In addition, waves along with currents cause significant bottom transport, which assists in bogging down wheels and tracks; and they have filled jetted

trenches before the cable can reach the as-excavated trench bottom. Waves can also build up, causing a disruption of surface support operations (Klopfenstein, 1974).

LOGISTICS SUPPORT. Small jetting systems are usually operated from boats of opportunity; the power supply and jetting vehicle are usually truck and air transportable. Larger systems usually have specialized boat- or barge-mounted seawater pumps for the jetting system and may have a specialized handling system for the vehicle. Operation of many jetting systems in high surf is ill-advised, especially since many systems require intermittent diver support.

WEATHER WINDOW. The weather window for jetting-in of a typical nearshore cable run would require between 1 and 5 days time.

HAZARDS. Cable in the nearshore is not normally buried by jetting because it cannot be jetted-in sufficiently deep in one pass to escape dragging anchors and scoring ice nor to escape exposure by scour. Added numbers of passes by jetting equipment are not normally justified economically. In those seafloor materials best trenched or fluidized by jetting, seafloor electrical cables are better protected by heavy armoring and weighting, causing them to bury themselves slightly below the seafloor surface due to wave action.

In those areas where cable damage due to dragging anchors or due to dredging for channel deepening is expected to be a problem, specialized jetting systems can accomplish seafloor penetrations of 30 feet (Welte, 1972); however, use of these deep-jetting, single-pass systems in the nearshore is usually not possible because of waves and swell. Multiple passes with a small jetting system to progressively bury a cable deeper and deeper are not possible in all nearshore environments, because eventually the sand replacement will outslip the jetting-excavation rate. No reported instance of electrical cable burial in the nearshore by multiple passes could be found in the literature.

DESIGN LIFE. Design life can be a significant variable; if less than 1 year, then burial by jetting would certainly not appear economically justified.

4.4.3 Dredging

Background and Description. Dredging for trench excavation includes a large spectrum of mechanical excavation concepts with or without suction applied for spoil removal.* This discussion will include

*This definition does not include the use of rock teeth on a cutter wheel or cutter ladder, usually used for cutting a deep, narrow slot in hard rock or frozen soil. This technique will be discussed in Section 4.4.5 Trenching.

all mechanical dredging or trench excavating techniques, even those using jetting to flush spoil from the mechanically excavated trench section. Plain suction dredging without mechanical assistance is also included in this section.

Trenching in the nearshore zone can be a difficult and expensive operation because wave action and currents act to backfill the trench as fast as it is excavated. Largely because of the problem of maintaining an open trench, electrical cables are usually fed to the excavation bottom by the trenching machine. If the excavation is done from a floating platform, however, the floating platform, moving in response to the waves, is not suited for the insertion of cables under most nearshore environmental conditions.

Dredging for cable burial in soils is usually reserved for the stronger, cohesive soils and for soils containing gravel or cobble; dredging is reserved for those soils not easily excavated by plain jetting (Section 4.4.2). The soil mechanically broken up by auger or wheel cutters and spoil removal expedited by suction or jetting. Dredging for trenching in rock is limited to softer and weathered rocks. In trenching for pipelines, either a large cutter head (36-in.-diam) or heavy grab bucket (8 ton) floating dredge is usually used. On these softer rocks, cables are usually protected by means other than trenching, although not always adequately. Attempts have been made to develop trenchers capable of cutting an adequate slot in medium hard (coral) to hard (basalt) rock, but so far these have not been entirely successful.

Procedure. The burial of electrical cable by a dredge-type trencher usually requires laying the cable first along the proposed route followed by trench excavation and cable insertion by the towed or self-propelled dredge trencher. Trenching is usually better accomplished starting at the sea end and pulling or walking the trencher into the beach.

Support Requirements and Installation Time Estimates. Support requirements and costs will vary considerably with the machine and the environment. For instance, one small cable burial device weighing 12,500 pounds and capable of burial depths to 4-1/2 feet in one pass and 7 feet in two requires: (1) a specialized diesel-hydraulic power unit weighing 6300 pounds; (2) a jet pump and air compressor (with size a function of water depth); (3) a towing winch of 20,000-pound capacity; and (4) a boat or barge capable of handling this equipment. Cable burial rates for this machine are: (1) hard clay - 8 ft/min, (2) hard packed sand - 10 to 14 ft/min, and (3) loose mud and silt - 25 ft/min. The cost of burial of 5,000 feet of cable to 4-1/2 feet was estimated at $25,000 in 1974 (Lynch, 1974).

Electric power to the machine is generally not used to minimize potential hazard to divers, who are required to initially position the machine with respect to the cable being installed, and who may be needed to monitor performance and to perform minor adjustments and repairs.

Selection Factors.

BOTTOM MATERIAL AND TOPOGRAPHY. The nature of the seafloor material and the thickness of the respective layers are very important when determining whether or not a given trench dredging system will perform as needed and when determining whether or not some less expensive cable protection system will be just as effective. For trench dredging systems, identification of gravel, cobble, and indurated deposits and near-surface rock is necessary to ensure cable burial.

WAVES AND CURRENTS. Waves and currents can have very great impact on the success of a cable burial project by dredging. Waves (1) can apply significant overturning forces to the jetting vehicle; (2) cause reduced seafloor bearing capacity and resulting sinkage of vehicle skids, wheels, and tracks; and (3) along with currents, cause significant bottom transport which assists in bogging down wheels and tracks and which can fill jetted trenches before the cable can reach the as-excavated trench bottom. Waves can also build up, causing a disruption of surface support operations (Klopfenstein, 1974).

LOGISTICS SUPPORT. Small dredging systems for cable or pipeline burial are usually operated from boats or barges of opportunity. All components of the smaller systems are usually truck and air transportable.

WEATHER WINDOW. The weather window for dredge-burial of a typical nearshore cable would require between 1 and 5 days.

HAZARDS. Dredge trenchers of the size used for electric cables are limited to about 8 feet of penetration; to reach that depth, normally two passes must be made. Burial to 8 feet will protect cables from damage by smaller dragging anchors but would not the larger anchors. A similar situation exists where ice scoring may occur. Thus, the potential for damage from dragging anchors and scoring ice masses should be evaluated when considering cable burial by dredging as protection.

DESIGN LIFE. Design life can be a significant variable; if less than 1 year, then burial by dredge trenching appears economically unjustified.

4.4.4 Explosive Excavation

Background. Successful cable stabilizaton requires either burying in the bottom or securing to it. In the case of hard rock or coral bottoms, a relatively smooth route must be provided that prevents bending the cable beyond its limits and provides adequate support to avoid long suspensions. If not naturally occurring, such a route can be prepared by either smoothing a path or by trenching and providing a relatively smooth trench bottom. In moderately hard materials, route

preparation may be feasible using mechanical equipment, such as a mechanical trencher (Section 4.4.5). In very hard materials or in extremely rugged topography in softer materials, explosive excavation may be the only viable route preparation option. While often considered as the last possible option, explosive excavation has been successfully used (and usually very poorly documented) in a wide variety of applications. In the past, explosive excavation has been coupled with additional techniques required to stabilize the cable in the blasted trench.

This section is based on Hallanger (1976) and is intended to provide sufficient background information on various blasting techniques to allow a preliminary assessment of the technical feasibility of utilizing underwater explosives in conjunction with cable stabilization operations. A complete review of the material presented in the above reference is recommended prior to final design implementation of any operation requiring the use of explosives.

Description. Explosive excavation in support of cable stabilization may be defined as the use of explosives to modify bottom topography to a contour and condition acceptable for the installation and stabilization of the cable. The use of explosives can range from very limited and simple applications involving a few contact charges placed by hand to extremely complex applications where large amounts of material must be moved and crawler drills are required to prepare the boreholes needed for proper explosive placement. Historically, the contact charge method has been used most often because it is simple and relatively quick. However, it does not provide a competent surface on the final grade and usually has dramatic environmental consequences. Drilled hole patterns, while requiring greater manpower, time, and skill, provide the ability to minimize environmental damage and provide much better controlled and competent final grade.

To design a blast or series of blasts to be used in a particular underwater explosive excavation job, a number of steps must be followed in an orderly sequence to develop the necessary supporting information on which the final calculations are based. These steps include definition of the problem; selection of the appropriate techniques; selection of the appropriate explosives; and calculation of the size, location, and delay patterns for the charges. In addition, two basic points should be considered throughout the design and implementation phases of any blasting operation: (1) the energy released by the explosive tends to follow the path of least resistance and (2) any job can be improved by observing the results of each blast and accordingly adjusting the charge size, placement, and initiation sequence.

PROBLEM DEFINITION. The initial definition of the problem is usually provided by the customer, who specifies the end result desired plus any restriction on methods. This specification normally includes the exact location of the job; the type of work to be performed; the time period in which the job must be completed; and, sometimes, information on the bottom conditions and water conditions at the site.

TECHNIQUE SELECTION. Consideration of the parameters discused in Chapter 2 along with the customer's requirements, will determine what combination of obstacle removal, trenching, and ramping will be required. Techniques available for use are limited only by the ingenuity of the blaster. The most common basic techniques range from simple contact blasting through complex drilled and delayed charge patterns. These basic techniques are discussed below.

1. Contact blasting - Contact basting is an external charge technique where the explosive charges are placed in intimate contact with the material to be excavated or broken. When used for general breakage and excavation in massive formations, the contact charge breaks the rock in a process called cratering. As the explosive detonates, it imparts a violent shock or blow to the material, much the same as a high velocity sledge. In a massive formation, this breaks the rock in a crater-shaped volume. When applied to smaller formations, such as a single rock or boulder, the result is a shattering of the rock. In soft porous materials, such as some types of coral, the explosive breaks by crushing rather than shattering. The resulting rubble is moved to a greater or lesser extent by the gas produced by the explosive.

The most effective explosives for underwater contact blasting are those with the highest detonation velocities. High gas production is also important if movement of the broken rock or coral is desired.

The efficiency of contact blasting depends on maintaining the intimacy of contact between the explosive and the material to be blasted. Cushioning material between the explosives and the rock, such as sand or water, attenuates the shock and thus reduces the breakage and effectiveness of the charge.

Experience has shown that the shape of the charge is also important. A conical pile, initiated at the top, produces the best results. Figure 4-56 illustrates a typical contact blasting charge placement for removal of a boulder.

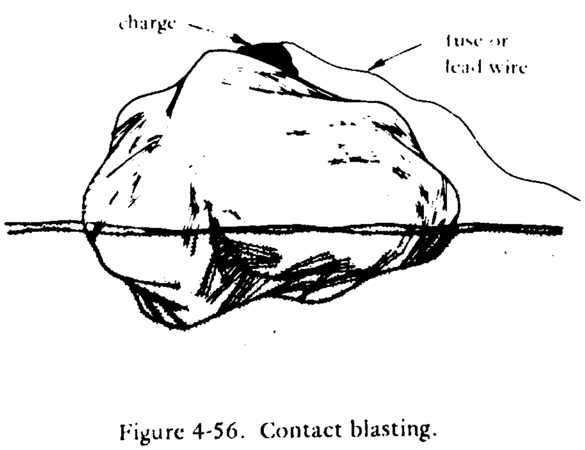


Figure 4-56. Contact blasting.

When the amount of explosive used per cubic yard of rock is the governing consideration, contact blasting is normally the least efficient technique. It also produces the greatest environmental damage because the unconfined charge releases a large part of its energy to the surrounding water rather than to the rock target. On the other hand, when

diver time is the limiting consideration, contact blasting is often the most efficient technique because it requires the least diver time to place the charges.

Since contact blasting breaks the material but often does not cause much material movement, a secondary system of removal must often be used. a hose charge or rope charge placed directly over the broken material will, upon detonation, cause the broken or fractured material to move outward. When the charge is initiated, the downward pressure causes the material to wash from the area below and adjacent to the charge. Repeated lineal charges of this type can, when the broken material has been removed, deepen and widen a trench effectively.

The use of contact charges can be severely complicated if waves, surge, or current effects are strong enough to move the charges from their desired locations. Weights or spike tie-downs may be used to secure the charges in their desired locations. Caution must be used to avoid having the tie-downs become unwanted missiles.

2. Snakeholing - Snakeholing, illustrated in Figure 4-57, is a blasting method used where boulders or rock formations are buried or partially buried in the bottom. It is a hybrid between external and internal charge techniques in that some confinement for the explosive is provided, but the explosive is still external to the rock being blasted. With this technique a hole is drilled under and immediately adjacent to the bottom of the boulder with a waterjet or mechanical device. A charge is then made up and placed in the hole, insuring that good contact is made with the boulder or section of rock involved. Intimate contact is required to insure that the explosive expends the maximum possible force and power on the boulder or rock.

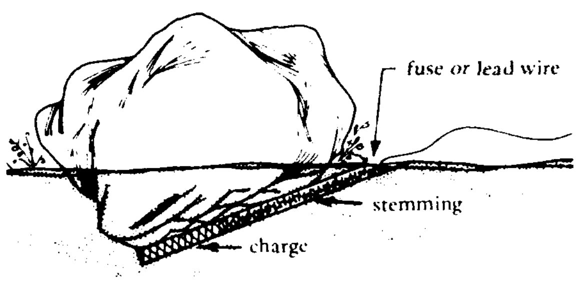


Figure 4-57. Snakeholing.

This is one of the simplest and most effective methods of boulder or rock section blasting where conditions permit its use. It must be remembered, however, that snakeholing is only efficient when holes can be drilled quickly and easily under the rock, and when intimate contact is made between the explosive and the rock or boulder to be blasted.

3. Blockholing - Blockholing, an internal charge technique based on drilling and loading one or more holes in a rock or ledge, is used primarily for obstacle removal. Figure 4-58 illustrates a typical blockhole charge. This technique consumes less explosive than those previously discussed because of the greater confinement of the charges and thus their higher efficiency. Blockholing does require the use of drilled holes which may be difficult or impossible to provide under certain conditions.

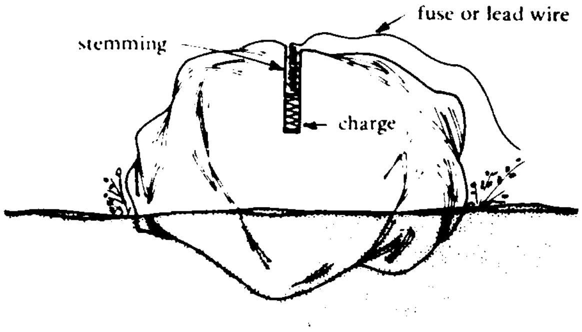


Figure 4-58. Blockholing.

The explosives will perform most efficiently if the holes are drilled so that each charge is close to the center of the volume it is to break. If fragmentation of the rock into small pieces is desired, the borehole should be oriented so that there is roughly an equal thickness of rock between the sides of the borehole and the rock surface. On the other hand, if the object is to split the rock with a minimum of fragmentation, the borehole should be located so the rock thickness between the borehole walls and the rock surface is much less along the desired split line than in other directions. Figures 4-59 and 4-60 illustrate these borehole orientations. Each hole should be stemmed and tamped well to prevent the explosive charge from rifling.

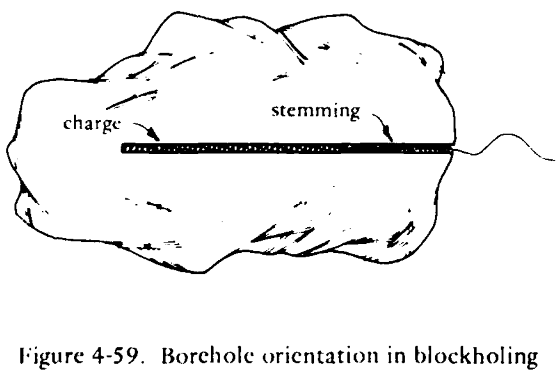


Figure 4-59. Borehole orientation in blockholing for maximum fragmentation.

4. Drilling Patterns - To obtain maximum results with precise control and minimum undesirable environmental effects, the classical method of blasting rock topside is to drill boreholes and load them with explosives. Properly utilized, this technique provides the most efficient and economical use of explosives in underwater blasting operations.

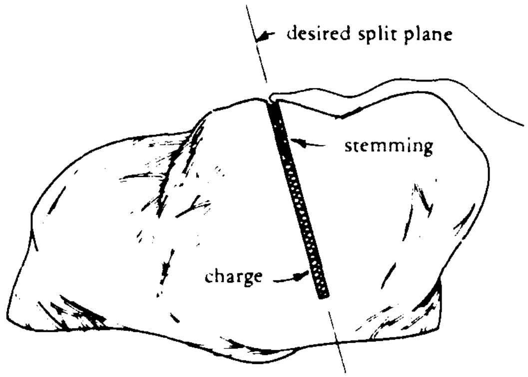


Figure 4-60. Borehole orientation in blockholing for splitting.

It is appropriate at this point to consider briefly how rock breaks when an explosive charge detonates inside a borehole. If the charge (and unstemmed borehole) is very short (no more than 2 or 3 borehole diameters in length), it can be considered a point charge. However, if the charge is long compared to the borehole diameter, it must be considered a line charge (see Figure 4-61). When the charge is initiated, a shockwave (detonation pressure) is transmitted into the surrounding rock, followed by the extremely rapid buildup of very high-pressure hot gases within the borehole. This normally happens so fast that the peak borehole pressure is reached before any significant rock movement can take place. The high bore pressure causes cracks to form radially outward from the charge. In the case of a point charge, these cracks propagate outward spherically. For a long borehole, the cracks propagate out perpendicularly from the borehole in all directions and give the borehole directional characteristics. As soon as some of the cracks reach a free face they begin to vent the hot gases and thus relieve the borehole pressure. These cracks also free the rock in the area between the borehole and the free face and allow the borehole pressure to begin moving it outward from the borehole. The amount of breakage and the shape of the crater depend entirely on the geometry of the borehole and the free faces and on the geology at the borehole.

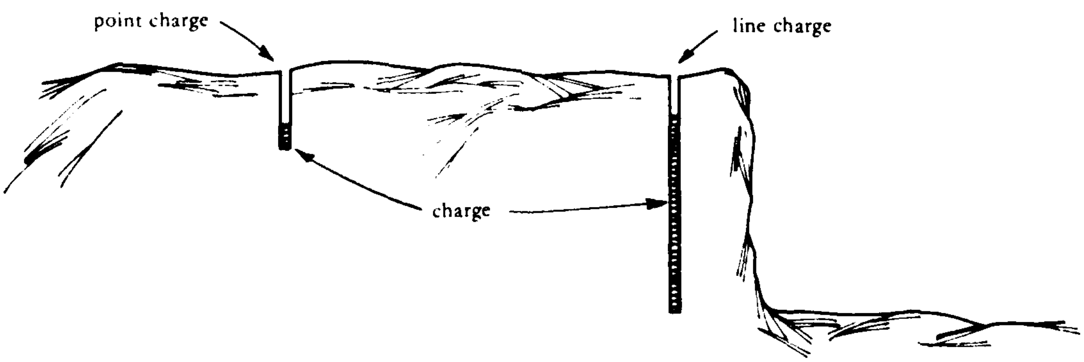


Figure 4-61. Point and line charges.

AD-A165 713 DESIGN AND INSTALLATION OF NEARSHORE OCEAN CABLE PROTECTION SYSTEMS(U) NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON DC CHESAPEAKE DIV 3/5

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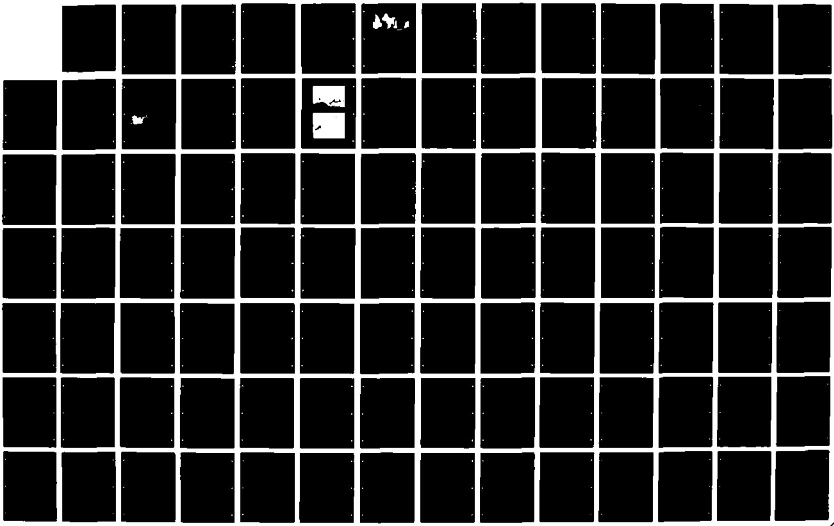
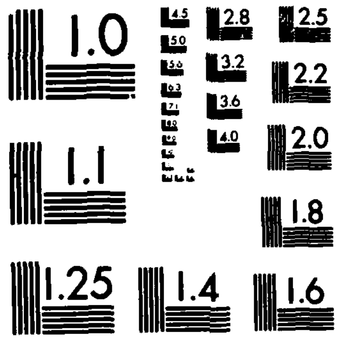




MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

Figure 4-62 illustrates typical breakage patterns for a single borehole with a variety of free face geometries. When more than one borehole is used, the relationship between adjacent boreholes must be considered. If delay patterns are also incorporated, then proper design requires that the free faces used in the design be those existing at the instant that the particular charge is initiated.

Figure 4-63 illustrates the breakage in the plane of the borehole axes, resulting from multiple rows of holes (as used in a quarry), where millisecond delays are used between rows.

In drilling a pattern of boreholes it is important that the position of the holes be maintained reasonably close to their theoretical or planned locations. The problem of hole location is greatly simplified if a cable or rope is laid as a centerline through the work area and secured in place. It may be marked at appropriate intervals to indicate the correct hole spacing. If possible, the hole locations should be marked individually with spikes, small lead or concrete clumps with fluorescent streamers, or other appropriate means.

When the hole has been bottomed and the drill rod and bit withdrawn, the hole should be loaded immediately unless a drill guide or stand pipe is left in place to keep the hole open for loading at a later time. When badly fissured rock is encountered it is sometimes necessary to blow the hole clean using an air wand or water jet.

If loading is done through a loading tube, the tube should be raised as the powder is pushed out the bottom. For underwater use the loading tube can be loaded with the complete charge for a hole, carried down and inserted into the hole. The cartridges are then pushed out in succession as the tube is withdrawn.

5. Shaped Charges - The use of shaped charges underwater for the excavation of rock and coral is a relatively recent development. Shaped charges provide deeper penetration than provided with conventional contact techniques for equal amounts of powder, yet give most of the advantages of easy charge placement with contact blasting. Charge placement patterns are similar to those used with drilled holes. Charges may be placed on the bottom individually or mounted in a wood or metal frame and dragged or lowered into position. Spacings and patterns for any particular charge and worksite combination should be determined by one or more test shots.

For a shaped charge to work underwater, it is necessary to keep the interior of the cone liner dry so that the jet has an air space in which to form. Shaped charge containers have been produced commercially that utilize about 50 pounds of liquid explosives for the charge (Johnson et al., 1971). Military shaped charges, such as the M2 and M3, can be successfully modified by using a container such as a tin can or plastic food freezer barrel

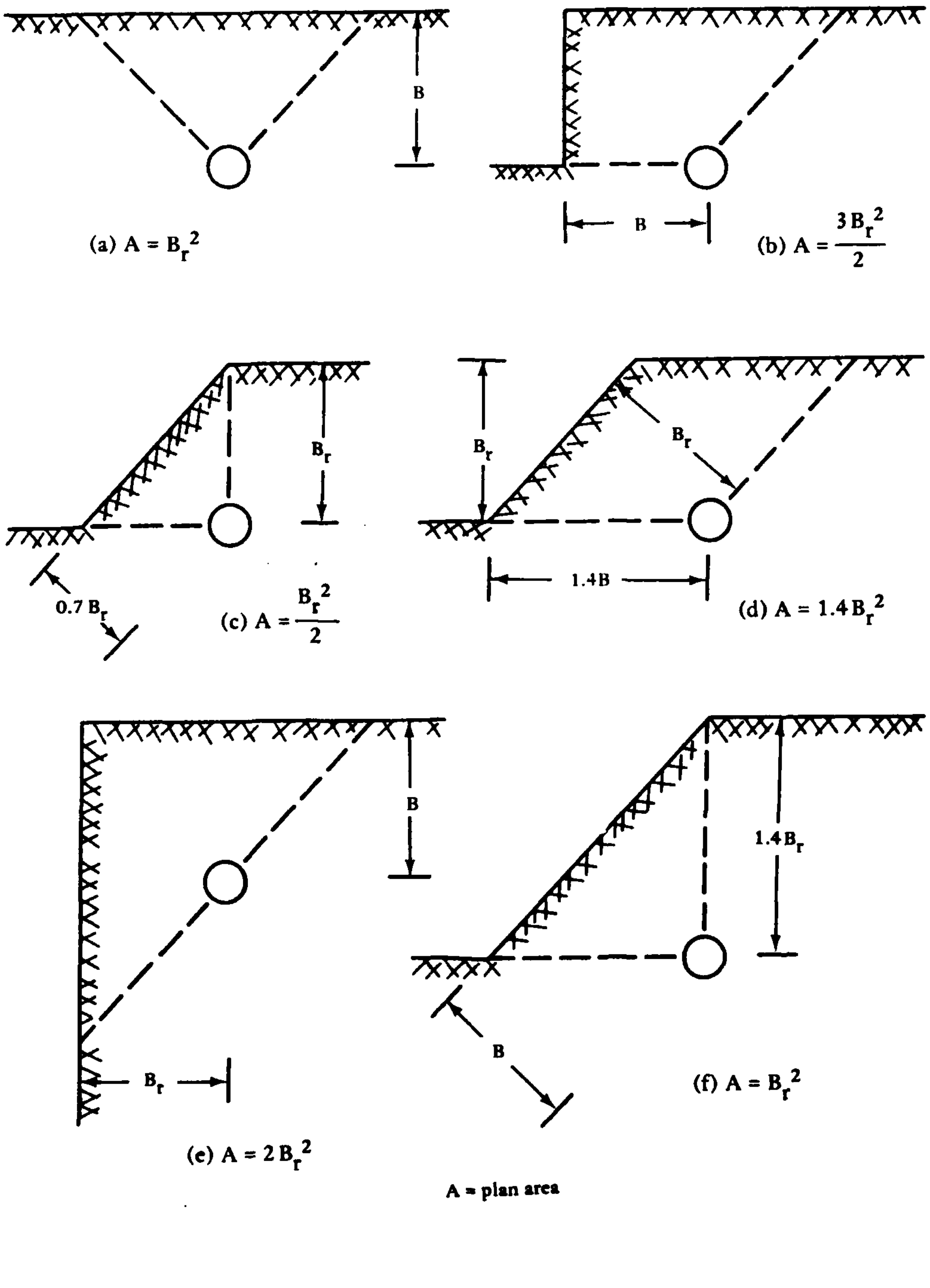


Figure 4-62. Basic crater forms for single charges in the plane of the charge diameter.

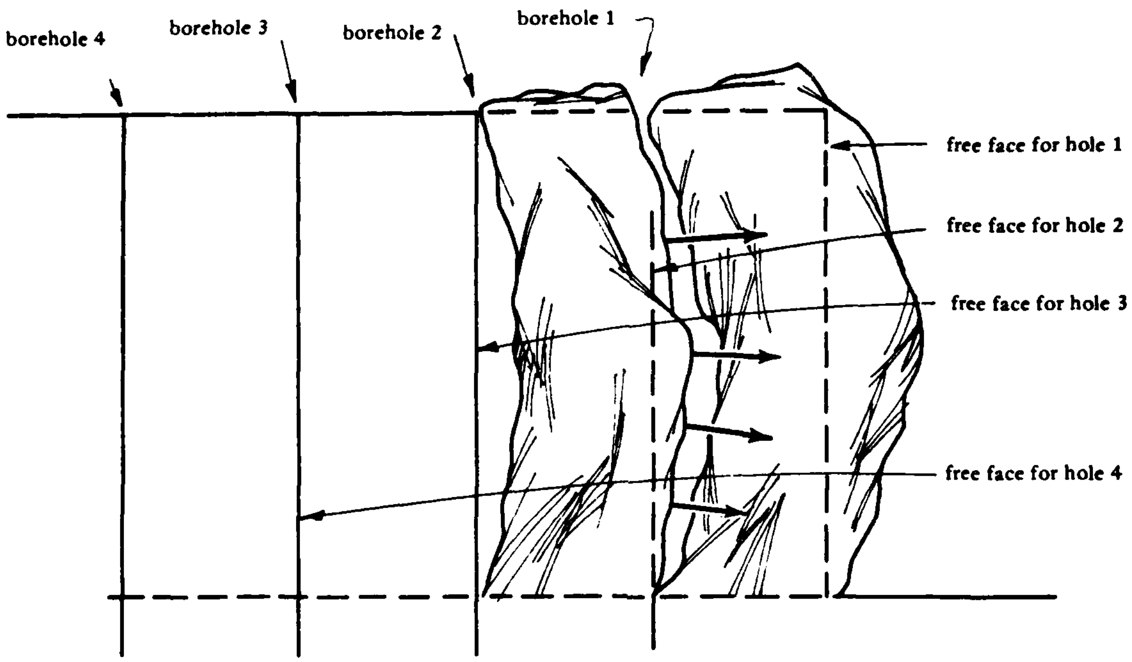
which just fits over the cone end of the charge. A thin layer of flexible waterproof sealing compound, such as a cold pouring bituminous material or RTV, is spread over the inside of the bottom and the charge inserted. More sealing material is poured or worked down between the charge and the container to provide a completely watertight seal. Small pieces of scrap iron, nuts and bolts, or lead shot are then poured into the space between the container and the charge as ballast to overcome the buoyancy of the air-filled cone. A final layer of sealing material secures the ballast in position. When the sealing material is set (or cured) the charge is ready to be used. Figure 4-64 shows the materials required to modify a standard M2A4 military shaped charge. It should be noted that a shrapnel problem may exist with this type of modification.

Procedure.

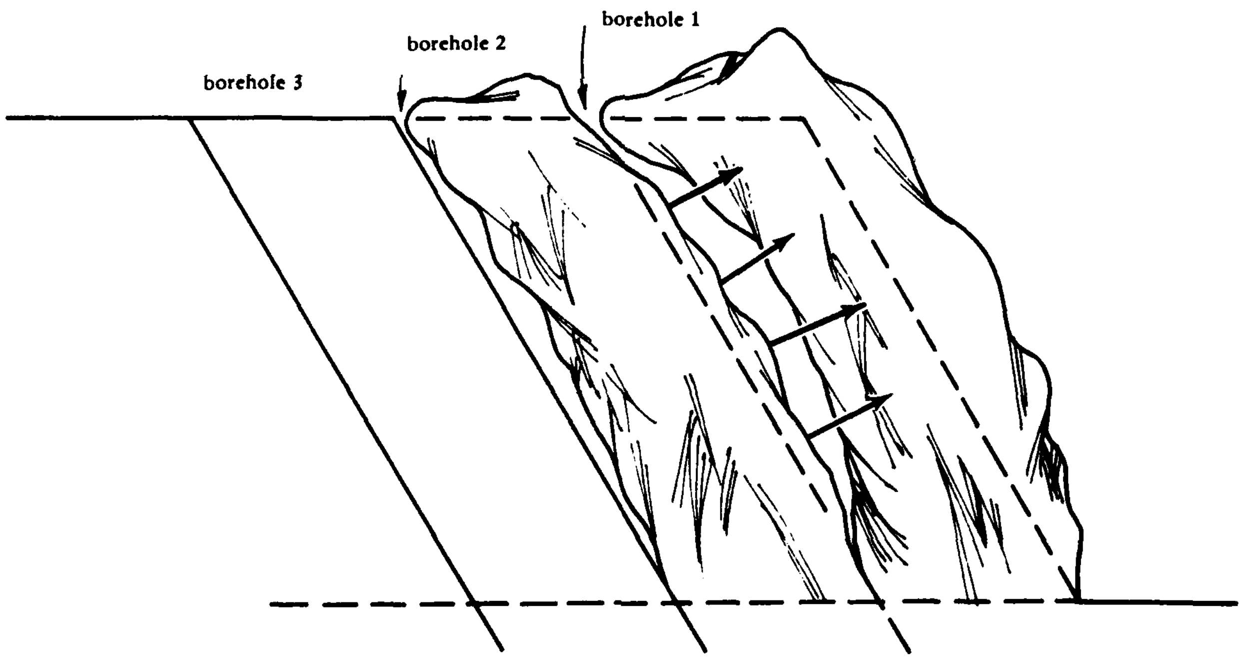
OBSTACLE REMOVAL. Underwater obstacles typically encountered along a cable route include coral heads, reef and rock outcroppings, and boulders. Natural obstacles come in many sizes and shapes: (1) rock obstacles may occur as long low ledges, massive lava flows, single pinnacles, low mounts, piles of boulders, or other formations; (2) coral occurs as reefs, large mushroom heads, low irregular masses, or other shapes. The primary objective in obstacle removal is normally to shatter the obstacle and move the rubble away from its original location. In some cases, such as removing coral heads, it may be acceptable merely to break the lower part of the head and tip it over.

In obstacle removal, the shape of the obstacle may often be used to advantage by proper charge placement: (1) pairs of charges placed on opposite sides of an obstacle, like "ear muffs" are effective in shattering the material; (2) charges placed on top tend to break the material and drive it down and away; (3) charges placed on opposite sides of a tall, pinnacle-like obstacle - one about half way up from the bottom and the other at the base - tend to topple the obstacle away from the upper charge.

TRENCHING. Trenching problems may require the use of any one, or combination of, the techniques described previously. The most commonly used are contact blasting and drilled patterns. Contact blasting, the traditional Underwater Demolition Team (UDT) technique, historically has used haversacks, Mk 8 hose charges, or Bangalore Torpedoes laid along the desired trench line. Haversacks, normally 20 pounds of explosive each, are placed in a row, or rows, spaced suitably to obtain the desired results; Hallanger (1976) suggests a spacing of 3 to 4 feet. Mk 8 hose charges and Bangalore Torpedoes are strung out (in triangular bundles if more than two lines are needed) to produce the trench.



(a) Vertical drill.



(b) Angle drill.

Figure 4-63. Breakage resulting from vertical and angle drilling using millisecond delays.

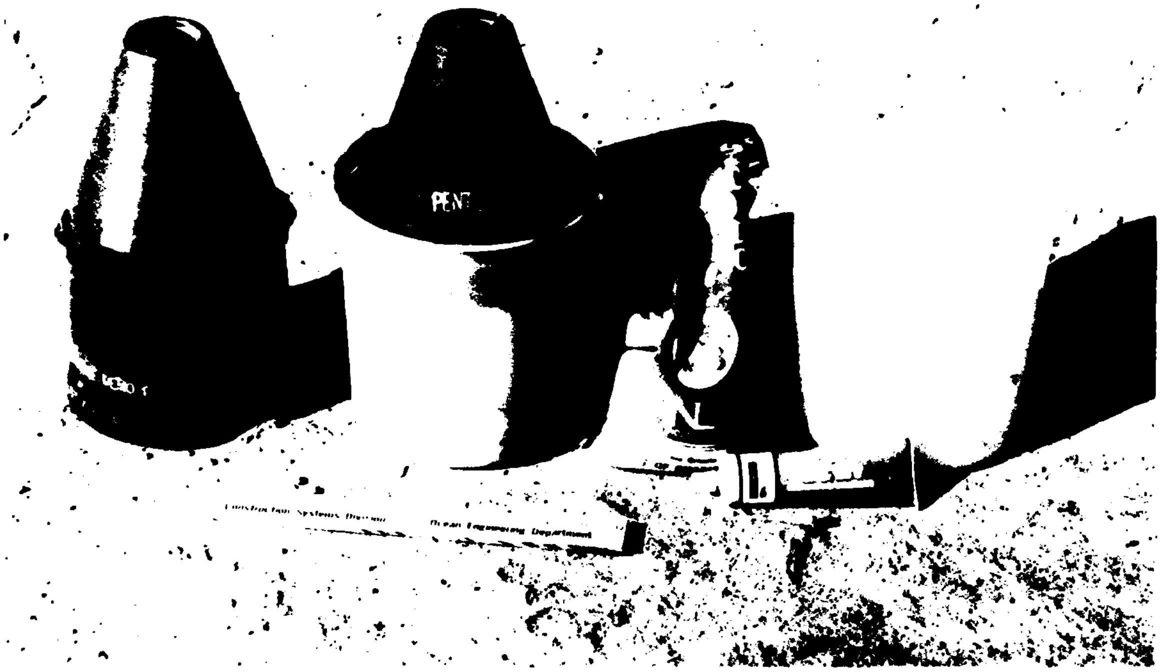


Figure 4-64. M2A4 shaped charge modified for underwater use and materials used to make the modification.

For narrow trenches, up to about 2 feet in width, a single row of drilled holes will often provide acceptable results. Spacing should be determined experimentally on-site, using the explosive selected for the job. Holes may be fired simultaneously or sequentially toward a free face using millisecond delays between holes.

Wider trenches, or extremely tough rock, or a requirement for vertical trench walls will require the use of two or more rows of holes. In this case the hole spacing (S_h) is always less than 1.4 B_r, and for relatively short holes is often chosen equal to the burden (B_r) for simplicity.

The holes in one row can be oriented in many different ways with the holes in the next row. The two simplest are the box and the diamond patterns. The diamond patterns is preferred where a smooth bottom is desired as it produces fewer ridges between hole locations.

An important part of planning the patterns is to determine what delays, if any, are to be used in the firing circuit. The use of delays allows greater control of the movement of the blasted

material and also reduces ground motion, water shock, and airblast intensities. Because an explosion tends to move the broken material toward a free face, the utilization of naturally occurring free faces or the creation of a free face with the first charges allows the blasted material to be moved in the desired direction. Delay intervals of about 50 msec between series of charges provides maximum effectiveness in moving rock underwater. Rock movement of distances greater than two or three times the burden dimension should not normally be expected for the muck pile. Complete clearing of all rubble will require additional use of hose or similar charges or mechanical clearing techniques.

In narrow excavations, such as trenches, initiating the charges at the outer edge or edges of the excavation with delays progressing across the trench will tend to move the rubble out of the excavation.

RAMPING. Excavation of a ramp may be necessary for a number of reasons; for example, a cable route may cross an underwater cliff or a road may be required underwater to move a track drill out to a work site beyond a ledge or dropoff. A ramp is simply a trench turned at an angle to provide a path up or down a cliff-type obstruction. Ramp depth compared to the adjacent bottom often varies considerably. Figure 4-65 shows an idealized ramp.

Two techniques are directly applicable to the ramping problem. In situations where time is the critical factor, contact blasting should be used. Figure 4-66 shows a typical charge placement and the approximate results of the blast for a contact charge shot. If a ramp wider than that produced by a single row of charges is desired, a checkerboard pattern should be used. Because the greatest volume of material must be blasted where the cut is deepest, the charge sizes will vary accordingly. Secondary blasting will often be required to level high spots and to remove the rubble; hose charges may be effective for the latter.

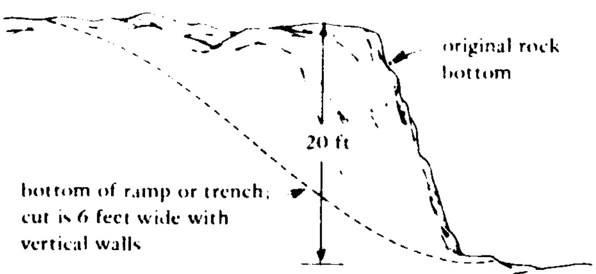


Figure 4-65. Typical idealized ramp for cable route.

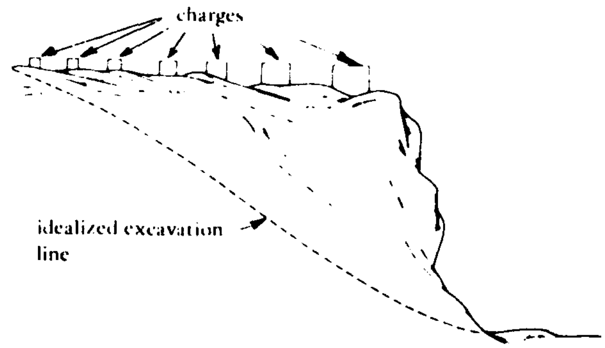


Figure 4-66. Ramp shot using contact charges.

The second applicable technique is the use of drilled patterns. The hole depth is adjusted so that the bottoms of the holes lie along the line of the desired cut as shown in Figure 4-67. These hole depths should be established using standard surveying techniques, if possible. In deeper water, measuring techniques must be improvised for the existing conditions of surge, current, and visibility. Delay patterns starting at the deep end of the cut and breaking toward the free face will be most efficient. Secondary blasting or other techniques will often be required to remove the rubble from deep narrow cuts.

Design Calculations.

DRILLED PATTERNS USING POINT CHARGES. The most recent developments in the area of explosive cratering for construction have been done by the United States Army Engineer Nuclear Cratering Group (NCG). They have been investigating the use of multiton charges for harbor excavation, railroad cuts, and similar jobs. The following information is based on the work of Johnson et al. (1971) and is interpolated to smaller charge sizes as necessary. A typical crater cross section is shown in Figure 4-68. The final geometry of a crater is a function of the burial depth of the explosive charge, as illustrated in Figure 4-69. Raw craters, or trenches, can be formed by placing the charges in a line and spacing them about equal to the apparent crater radius for a single charge. Parallel rows of charges with a row spacing of about 1.5 times the apparent single crater radius and an initiation delay between rows of 25 to 50 msec can be tried if a wider trench is desired.

Crater dimensions for topside excavation are reasonably predictable. Experimentally obtained results for relatively large cratering charges in dry rock have given the results shown on the right side of Figure 4-70. Interpolation of these data for smaller charges is shown on the left side of the figure. In underwater work, the water overburden has a major influence on the crater characteristics. The in-rush of displaced water after the detonation redistributes ejecta and may wash in material which would otherwise remain in place. Thus, there are no reliable scaling relationships for predicting the size and geometry of underwater craters. Although the crater radius scales well when the water depth is a fixed fraction of the total charge depth of burial, the crater depth may not scale similarly.

To make practical use of underwater cratering, test shots must be performed to determine crater geometry under the particular conditions at the site.

As a first approximation, the apparent underwater crater radius is assumed to be the same as the apparent radius for a land crater in similar material, and the apparent depth is half that of the land crater. True crater dimensions should approximate those

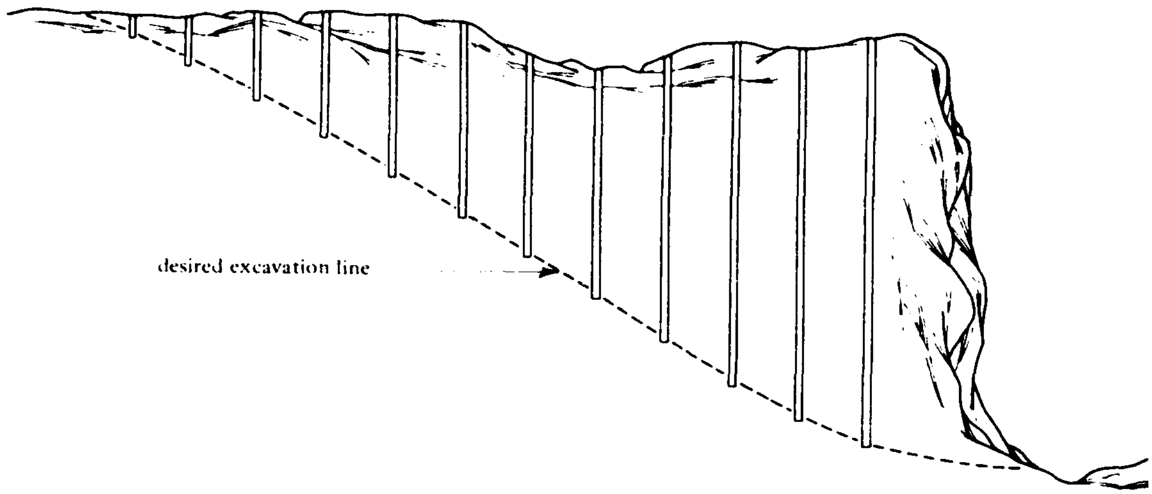


Figure 4-67. Ramp shot using drilled holes with constant spacing and burden.

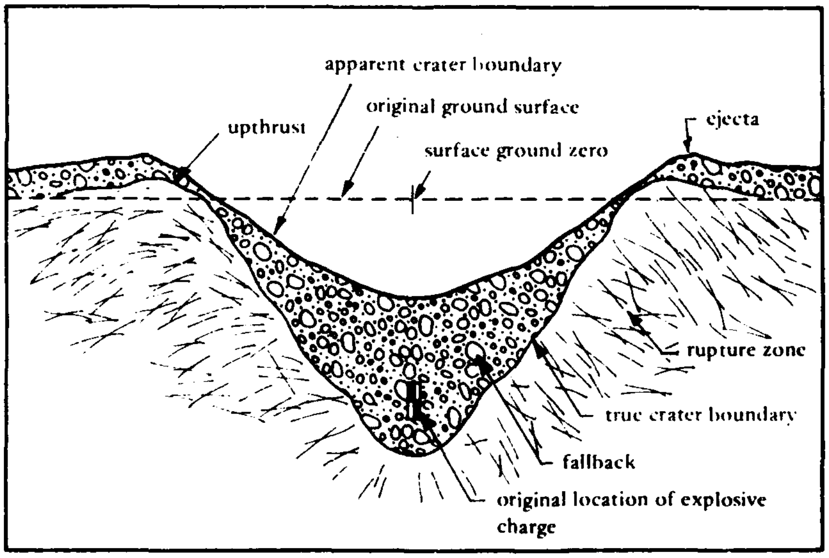
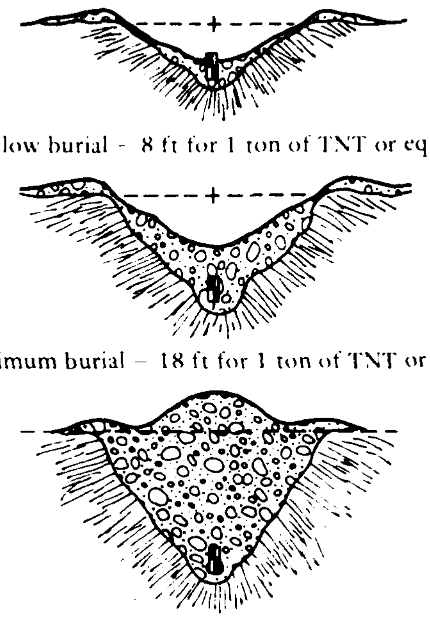


Figure 4-68. Cross section of typical crater in rock, showing nomenclature.



(a) Shallow burial - 8 ft for 1 ton of TNT or equivalent

(b) Optimum burial – 18 ft for 1 ton of TNT or equivalent.

(c) Deep burial 28 ft for 1 ton of TNT or equivalent.

Figure 4-69. Crater profiles for various depths of burial.

of land craters in similar materials. In determining charge depth of burial, the water layer may be regarded as a layer of the bottom material having a thickness equal to one-half the water depth.

DRILLED PATTERNS USING LONG BOREHOLES. The four basic parameters from which all others can be determined are listed as follows: (1) burden (B_r) - the distance between the borehole and the nearest free face at the instant of initiation; (2) diameter of the explosive charge (D_E); (3) density of the explosive (ρ_E); and (4) density of the rock (ρ_R) (see Table 4-19). Rock density will range from about 1.7 to 3.2 gm/cc with an average value of about 2.6 gm/cc; values for coral may be significantly less. The explosive used is often determined by what is readily available, and thus the explosve density is not easily varied. In addition, most explosives have densities varying between the narrow limits of 0.87 to 1.6 gm/cc, with the higher densities more desirable for underwater work. Borehole diameter, and thus the explosive diameter, is controlled by the drilling equipment available. For underwater work this normally limits boreholes from 1 to 4 inches in diameter. Thus, it is seen that there is little or no control over the rock and explosive densities, and only a moderate control over explosive charge diameter. The values of these three parameters determine the burden to be used according to the equation.

$$B_r = 37.8 D_E \left(\frac{\rho_E}{\rho_R}\right)^{1/3} \qquad (4\text{-}5)$$

Spacing (S_h), the fifth important parameter, is always $\leq 1.4\ B_r$ and approaches B_r as the borehole length decreases.

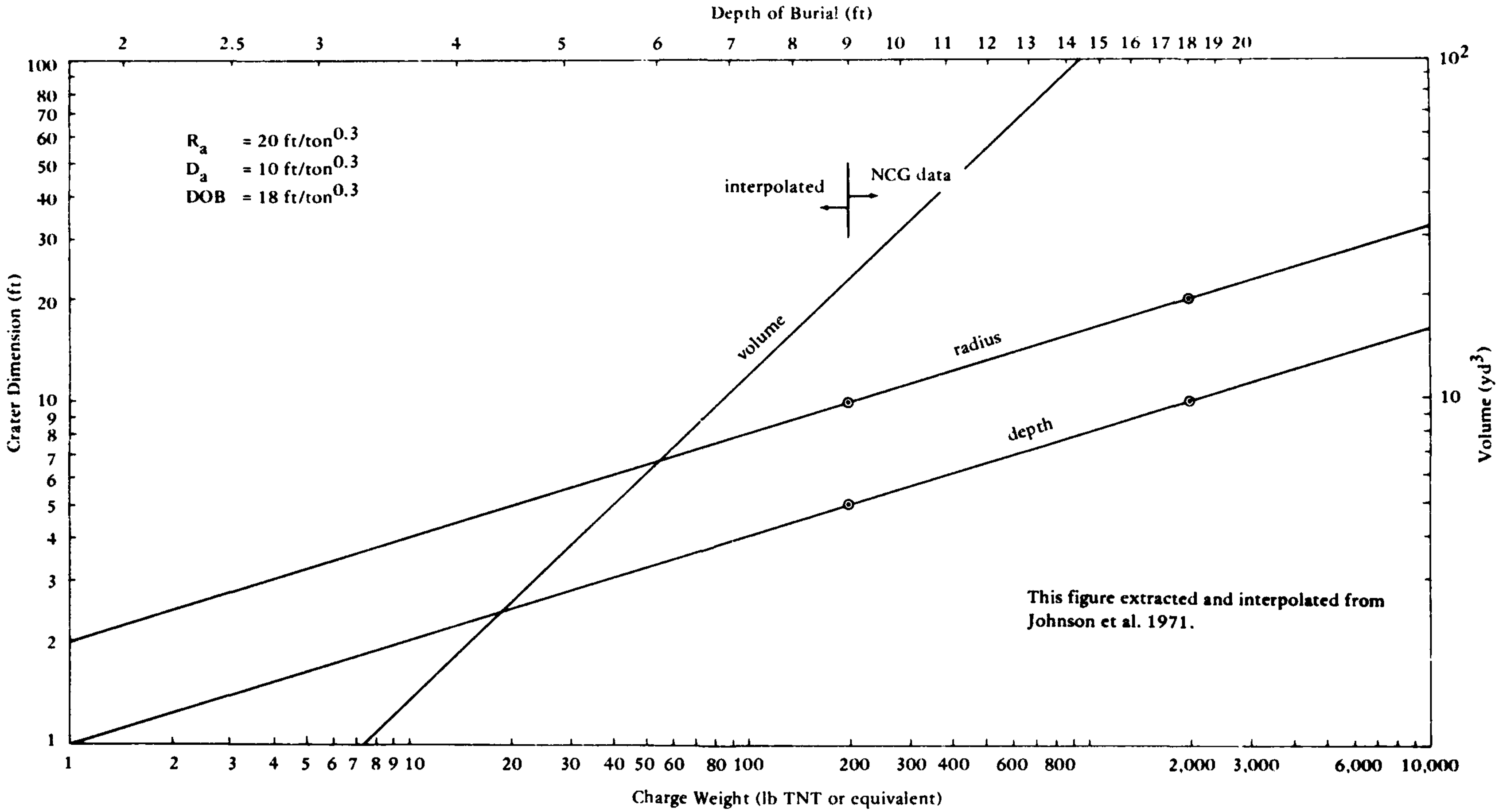


Figure 4-70. Crater parameters for dry rock.

Other factors which must be considered but that cannot be easily put in numerical form include the following:

(1) Water depth has the effect of increasing the volume of material that must be moved by a given charge. This can also be interpreted as making the effective explosive charge diameter smaller, or as decreasing the allowable burden. In any case, the amount of decrease in the allowable burden must be determined experimentally by test shots on site.

(2) Excavation depth will determine borehole depth. The need to keep the burden and spacing less than or equal to one-half the hole depth will sometimes dictate the use of small diameter charges.

(3) Detonation velocity of the explosive is of importance only when the rock being blasted is both massive and unfractured; then a high detonation velocity is desirable.

EXCAVATION USING CONTACT EXPLOSIVES. Figures 4-71 through 4-73 give typical trench profiles for contact charges in coral. To obtain similar information for the rock at a given work site, one or two test shots should be made, and the resulting trench versus the number of charges per unit length can then be scaled proportionally.

Support Requirements. Manpower and equipment may be either a dependent or independent variable. If limitations are placed on either then they may dictate time requirements or the type of technique which can be employed. Likewise, other factors such as the weather window or environmental constraints may make certain minimum manpower and equipment support levels mandatory.

Table 4-19. Density and Relative Toughness of Rocks

Kind of Rock	Density		Relative Toughness[a,b] (Limestone = 1)
	gm/cc	lb/ft^3	
Andesite	2.4 to 2.8	150 to 175	1.1
Basalt	2.4 to 3.2	150 to 200	1.7 to 2.3
Conglomerate	2.0 to 2.6	125 to 162	NA
Dioritic	2.5 to 3.2	156 to 200	1.9 to 2.1
Felsite	2.4 to 3.2	150 to 200	NA
Gabbroic	2.7 to 3.2	169 to 200	NA
Gneiss	2.4 to 2.9	150 to 181	1.0 to 1.9
Granitic	2.5 to 3.1	156 to 194	1.5 to 2.1
Limestone	1.7 to 3.0	160 to 187	1.0
Marble	2.1 to 2.9	131 to 181	NA
Quartzite	2.0 to 3.2	125 to 200	1.9 to 2.7
Sandstone	2.0 to 3.1	124 to 194	1.5 to 2.6
Schist	2.4 to 2.8	150 to 175	1.0 to 2.1
Shale	1.8 to 3.1	112 to 194	NA
Slate	2.5 to 3.1	156 to 194	1.2

[a] Found in Energy of Explosives and Toughness of Rock in Selecting Explosives, by W. O. Snelling, Eng & Con, Jan 8, 1913.

[b] NA = not available.

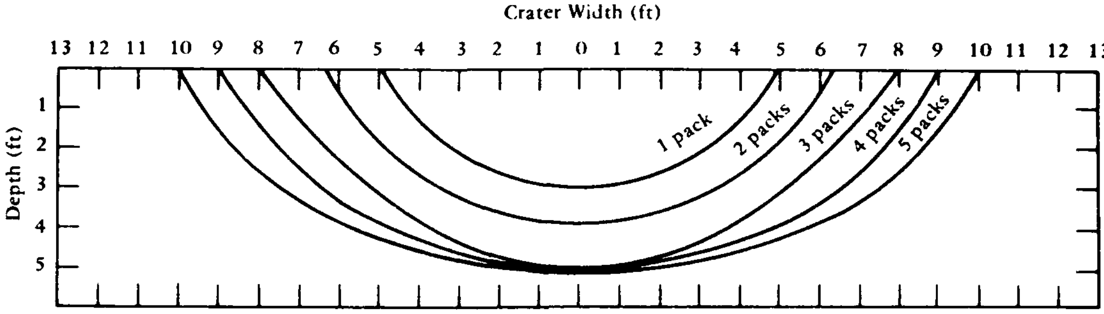


Figure 4-71. Comparative depth and width of cuts made in coral by 20-pound haversacks of plastic explosive.

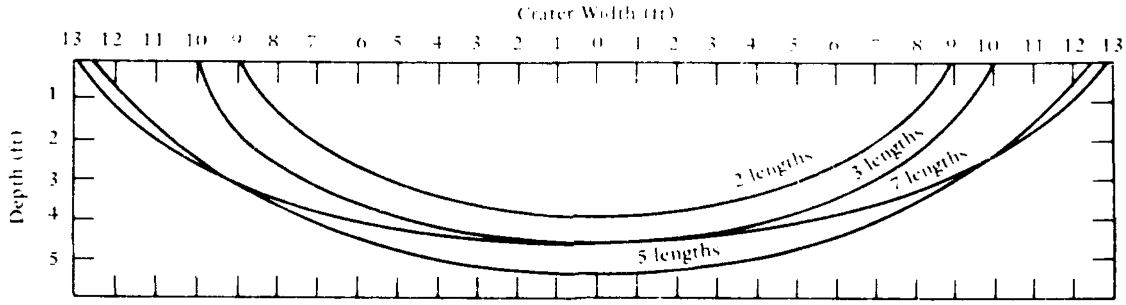


Figure 4-72. Comparative depth and width of cuts made in coral by demolition charge Mk 8.

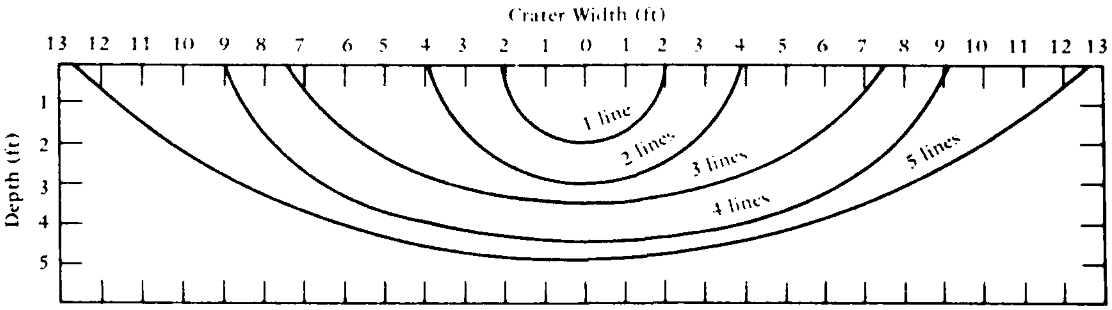


Figure 4-73. Comparative depth and width of cuts made in coral by Bangalore Torpedos.

MANPOWER.* Minimum crew for any underwater explosive excavation job is the number required to conduct diving operations; working divers must be qualified to handle explosives. As constraints are imposed on the time allowed to do the job and the technique to be used, manpower requirements will increase rapidly.

EQUIPMENT. Minimum equipment for any underwater explosive excavation job (required for the explosive portion of the operation) includes the explosives, detonating cord, caps and a means for initiating the caps, necessary safety equipment, and a secure place for storage (see Hallanger, 1976, Section 6.2 for further details). Constraints which require bockhole or drilled pattern techniques also require the drilling equipment. Both pneumatic and hydrualic drills have been used to produce the necessary hole patterns. Recently developed hydraulic drills (Brackett and Tausig, 1977) are usually preferred, however, because they do not produce the percussion effect associated with pneumatic drills. If hole diameters greater than 3-1/2 inches are required, a modified pneumatic tracked drill (Page, 1973) should be considered for borehole production. The amount of equipment also depends on crew size and time constraints.

Installation Time Estimates. The time required for any installation will be highly specific for site and crew. In addition to the information given earlier, it is also true that the more experienced and skilled the work crew is, the faster it will be able to do the job. It must also be recognized that if the environmental and topographic conditions are severe enough to require explosives, then working conditions are probably going to be extremely difficult and progress slow. Rates of progress for route preparation can vary from a few hundred feet per day for a minimum crew using hose charges in relatively soft materials such as coral to a few feet per day for each track drill in very hard rock combined with rough bottom topography and wave, current, or visibility problems. Many examples can be found where the job is not possible under normal weather conditions, and divers can work effectively only during rare weather windows. The best source of guidance on time requirements is an experienced diver-blaster's firsthand opinion, based on an on-site inspection and adequate environmental data.

*The information provided in this section is based on limited data from a few previous installations and as such is intended only as a guide in developing preliminary cost estimates. Since environmental conditions vary considerably from site to site the final decision on the number of personnel required to safely conduct the operation must be left to the discretion of the diving officer.

Selection Factors.

BOTTOM MATERIAL AND TOPOGRAPHY. This parameter will affect the choice of techniques and also the choice of explosives. Extremely soft materials such as mud are not readily moved by explosives.

WAVES AND CURRENTS. Waves and currents are a limiting factor in that they limit the amount and type of work a diver can accomplish. They can also move charges after placement or cause problems with detonating cord lines. Diving operations involving drilling and handling explosives in current and surge greater than 1/2 knot are difficult and usually dangerous.

LOGISTICS. The logistics available to support a given site can drastically limit the selection of an explosive excavation technique. For example, lack of roads can prohibit the use of heavy machinery such as track drills. It can also limit the amount of explosive that can be brought to the site. Lack of magazines can severely limit the amount of explosives that can be kept near the work site or can force the use of more expensive field-mixed explosives.

WEATHER WINDOW. If weather conditions are such that divers can only work effectively during short weather windows, then the duration and frequency of occurrence of this window dictates the time available to do the job. This in turn may dictate which technique is acceptable.

VISIBILITY. The lack of underwater visibility severely restricts the ability of working divers to select the optimum route and to work effectively along that route and directly affects the time required to complete the job.

HAZARDS. Unless a narrow deep trench with nearly vertical sides can be produced along the entire length of the cable, then explosive excavation provides relatively little protection for the cable against the environmental hazards discussed in Chapter 2. Since the production of such a trench is costly and time-consuming, explosive excavation in the past has been limited to route preparation with other techniques used to stabilize the cable.

WIND. Wind is not a limiting factor except for its effect on topside support operations.

DESIGN LIFE. Along with the environmental situation, design life requires a competent surface on which to secure the cable. In such a case the use of drilled patterns is mandatory, except in very unusual situations.

LENGTH OF PROTECTED CABLE. Time availability, manpower availability, and environmental restrictions indirectly influence the length of protected cable.

ENVIRONMENTAL RESTRICTIONS. In many areas the use of explosives is restricted or forbidden because of potential environmental damage that may result. As restrictions become more severe the options are reduced to use of drilled patterns with hole-to-hole delays to minimize the amount of explosive actually detonating at any one instant and so to minimize the environmental effects.

4.4.5 Mechanical Trenching

Background and Description. Two previous Navy cable installations have utilized a mechanical trencher to provide a narrow slot in the seafloor for stabilization of the cable. Although both projects fell short of providing the desired length of trench, they were successful in demonstrating that the concept of mechanical trenching on the seafloor is feasible.

In 1975, a Vermeer T600 disc saw trencher was modified by removing the diesel engine and transmission and replacing it with hydraulic motors powered by a remote diesel-hydraulic source. The remote power source utilized a turbocharged 200-hp diesel to drive a pressure-compensated piston pump capable of delivering 105 gpm at pressures up to 5,000 psi. The 7-ft-diam disc saw was capable of cutting a trench 31 inches deep by 6 inches wide.

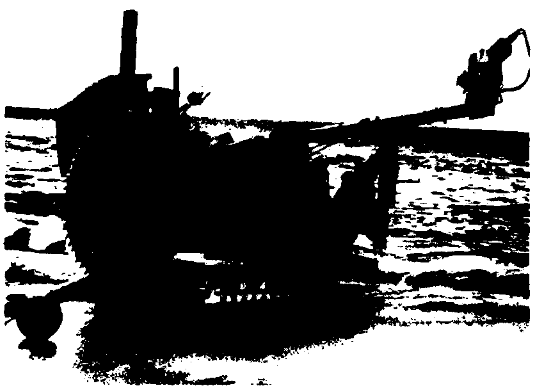


Figure 4-74. Seafloor trencher.

In August 1975 the modified trencher (Figure 4-74) was deployed to Midway Island to be used during a cable installation operation. Technical problems were encountered during the 30-day operation which reduced trencher performance to considerably less than originally projected. A total of 600 feet of trench was cut in a protected lagoon with bottom conditions ranging from sand to solid coral.

During the following year the trencher was rebuilt for use during a cable installation in the Hawaii area. This second-generation underwater trencher utilizes the same chassis, track-drive system, and disc saw as that used at Midway. However, the entire hydraulic system utilizes separate flow control valves for each track and for the cutter wheel motor, allowing much better control of the power distribution to the various fluid power circuits.

In early September 1976 the trencher was shipped to the Pacific Missile Range Facility, Barking Sands, Kauai, to be used to provide trenches for three list 5 SD coaxial cables. During 3 weeks of operation, approximately 400 feet of trench was cut through beach rock on land before mechanical failure caused cancellation of the trenching operation.

In general, the trencher appeared to operate much better than its Midway predecessor. The major problems which plagued the trenching operations at Midway seemed to be solved (i.e., hydraulic hoses rupturing, inadequate control of the fluid power distribution, and jamming of the track-drive system). The problems which caused the trencher to produce less than the desired results were primarily a result of the environmental effects (heavy surf) on various mechanical systems (Brackett et al., 1976).

Power for the trencher was supplied by a diesel-driven, pressure compensated, variable flow pump. While operating the trencher on land, either the maximum pressure had to be held to less than 2,000 psi or the power source had to periodically be shut down to allow the oil to cool off.

Because of this power limiting factor plus the power loss through the 300-foot hydraulic hoses, it is estimated that when operating continuously onshore, less than 50 hp was being delivered to the trencher. During submerged operation it was expected that sufficient cooling would be produced by the seawater to allow the power to be increased by at least 100%.

Additional problems experienced with the power system included:

(1) Pressure oscillations at the pump caused by the accumulator effect of the high-pressure hose and the slow response time of the pressure compensation system. These fluctuations were often in the range of 2,000 psi and lasted for several minutes before they were damped out.

(2) The weight of the hydraulic hose which poses a potential safety hazard to people working with it on the beach.

A 31-inch-deep by 6-inch-wide trench can be cut by the 7-ft-diam disc saw. One hundred thirty carbide-tipped self-sharpening teeth are distributed around the perimeter of the heel. The maximum continuous rotational speed developed was about 23 rpm (approximately 500 surface ft/min), which is only about one-half the speed required for rock cutting.

The material trenched on the beach ranged from soft sandstone to a hard cementitious sandstone. Advance rates varied from 2 ft/min to about 0.1 ft/min, respectively. Except for the relatively slow advance rate the trenching mechanism seemed to work well. Two problems were observed: (1) while attempting to cut a 50-ft-long curved section of trench in the harder rock, 28 cutting teeth were broken, and (2) sand buildup in the cutter wheel drive sprocket caused the drive chain to stretch and jump on the sprocket several times, which ultimately led to failure of the triple roller chain.

The track system has not been changed from that used at Midway except for use of higher torque drive motors which did allow the trencher to climb steeper slopes. The problems of low ground clearance

and poor obstacle negotiation still exist. Even with the added power to the track drive, it is impossible for the trencher to negotiate any vertical discontinuity greater than 6 inches in height.

The combination of low ground clearance and high bearing load of the tracks presented problems in developing sufficient drawbar pull in the surfzone. While trenching in the surfzone, the tracks sank into the sand to such a depth that the trencher would bottom out on the skid pan thus stalling the trencher. The trencher then had to be lifted using the cutter wheel and scraper blade to allow sand to fill back in under the tracks thus further reducing the average trenching speed.

The rigid track suspension also posed some problems since on irregular surfaces at least one of the tracks was usually suspended for most of its length. This reduction in contact area resulted in increased track slippage.

Even with the limited success of these previous attempts, interest in this technique of cable stabilization continues because it offers the greatest potential for protection of cables against hydrodynamic forces and most of the hazards discussed in Chapter 2. An analysis by NAVFAC of cable stabilization operations concluded that for cable lengths greater than 1,200 feet, the mechanical trencher is more economical than the conventional split-pipe operation.

Based on this, a study conducted by Brackett et al. (1977) resulted in the conceptual development of a mechanical trencher for nearshore cable stabilization operation which would not suffer from problems previously experienced. A scale model of this concept is shown in Figure 4-75. The trencher system consists of four basic modules: (1) the trencher, (2) a diesel-driven electric generator, (3) a beach cable reel, and (4) a beach control station.

The trencher consists of a single pair of tracks, each 3.6 feet high by 4 feet wide by 18.25 feet long. The chassis is rigidly attached to one track while the second track is allowed to pivot. The assembly is 14.6 feet wide in its operational configuration. Mounted on the chassis are the trenching mechanism (aft), the power cable reel (center), and power conversion pressure housing (forward) (see Figure 4-76). This housing contains the electric motor, the four main hydraulic pumps, and any electrically actuated valves required for the control system.

The electric generator is rated at 450 kW and is powered by a 660-hp diesel engine. The unit is 8 feet high, 5 feet wide, and 12-1/2 feet long.

A small power cable reel is located on the beach to eliminate the problem of cable overheating when trenching on land. The 4x5x5-foot reel holds up to 500 feet of power cable.

The beach control station contains all of the monitoring and controls necessary for remote operation of the trencher. Exact size requirements have not been established, but it is estimated that an 8x8x10-foot shelter will hold the necessary equipment and operators.

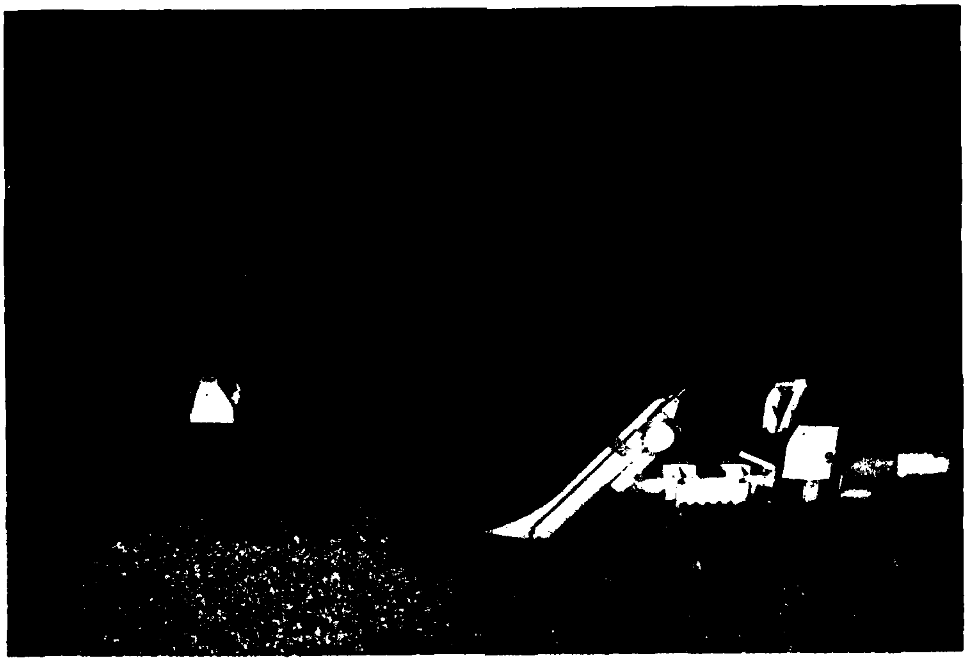


Figure 4-75. Concept model of nearshore cable trenching system.

Figure 4-76. Concept model of nearshore cable trencher.

The operational weight of the trencher will be 97,400 pounds. To meet the logistics constraint of a maximum shipping weight of 70,000 pounds per single unit, the trenching mechanism and power cable and reel will be removed from the trencher. This results in a shipping weight of 70,400 pounds for the trencher with an additional 57,900 pounds of hardware shipped separately.

The anticipated cost to acquire such a piece of hardware was estimated at approximately $1,000,000 which included $822,000 for design and fabrication and $185,000 for testing, training, and maintenance prior to the first deployment.

Because of the high cost, the procurement of a nearshore trencher could not be justified until sufficient cable installations were planned to amortize the expense.

Procedure. The primary operating scenario calls for deployment of the trencher and its support equipment to the beach site by truck. At the beach site, the equipment will be assembled and prepared for operation. The control and power modules will be positioned on the beach and connected to the trencher by the power and control tether. The trencher will enter the surf and, at the initial point on the cable-pipe track, begin trenching operations. The trencher then will dig along the cable track under the control of the operator on the beach. Deployment or operational control of the trencher from a surface support platform will be a project option dependent on the environment and length of burial track at the project site.

All normal operations of the trencher will be automatic or remotely controlled and monitored from the beach site. All operations will be along presurveyed and cleared (to within the operating limits) rights of way. The trencher will cut an open trench 12 inches wide by 3 feet deep in rock (or by 7 feet deep in sand), into which pipe or cable may be subsequently laid subject to removal of loose spoil materials as a separate operation. However, provision has been included for simultaneous digging and laying functions (i.e., the cable will have already been laid along the track, and the trencher will under-run and lay the cable as it trenches).

Divers will not be used or allowed near the trencher during trenching operations because of potential electrical shock. They will be used only to assist the beach operator in the initial positioning of the trencher or to verify the success of the trencher cutting operations. In the event of trencher malfunction, divers will be used to assist in recovery operations.

Support Requirements.

MANPOWER. Because of the sophistication of several of the trencher subsystems, it is anticipated that a dedicated crew will be required for operation and maintenance. The minimum crew should consist of personnel qualified in the following disciplines:

Personnel	Requirements
Electronics	1
Electrical Power Systems	1
Hydraulics	1
Mechanical Systems (diesel engine)	1
Operators	3

In addition, a minimum of six divers are required for route survey and preparation, underwater maintenance of trencher components (cutter bits, etc.), and recovery in the event any of the critical subsystems cannot be repaired underwater.

EQUIPMENT. In addition to the trencher system components previously discussed, the following equipment and consumables are required to support the operation.

Equipment	Requirements
Diesel Fuel	220 gal/day
Hydraulic Oil	as required
Trencher Chain	1
Cutter Bits	90
Cutter Bar	1
Hydraulic, Electric (miscellaneous spare parts)	as required
Diving Gear	as required
Portable On-Site Maintenance Facility	1

Installation Time Estimates. Based on a constant power density of 40 hp/ft^2 of cutter contact area as reported by Brackett et al. (1977), the advance rate of the trencher will not be affected by the size of the trench (as long as sufficient power is available to the cutter for the largest trench anticipated). Advance rate will be affected by the strength and type of seafloor material encountered. Figure 4-77 presents theoretical speeds of advance for a trencher producing a 1x3-foot trench. Based on experience from terrestrial trenchers, excessive

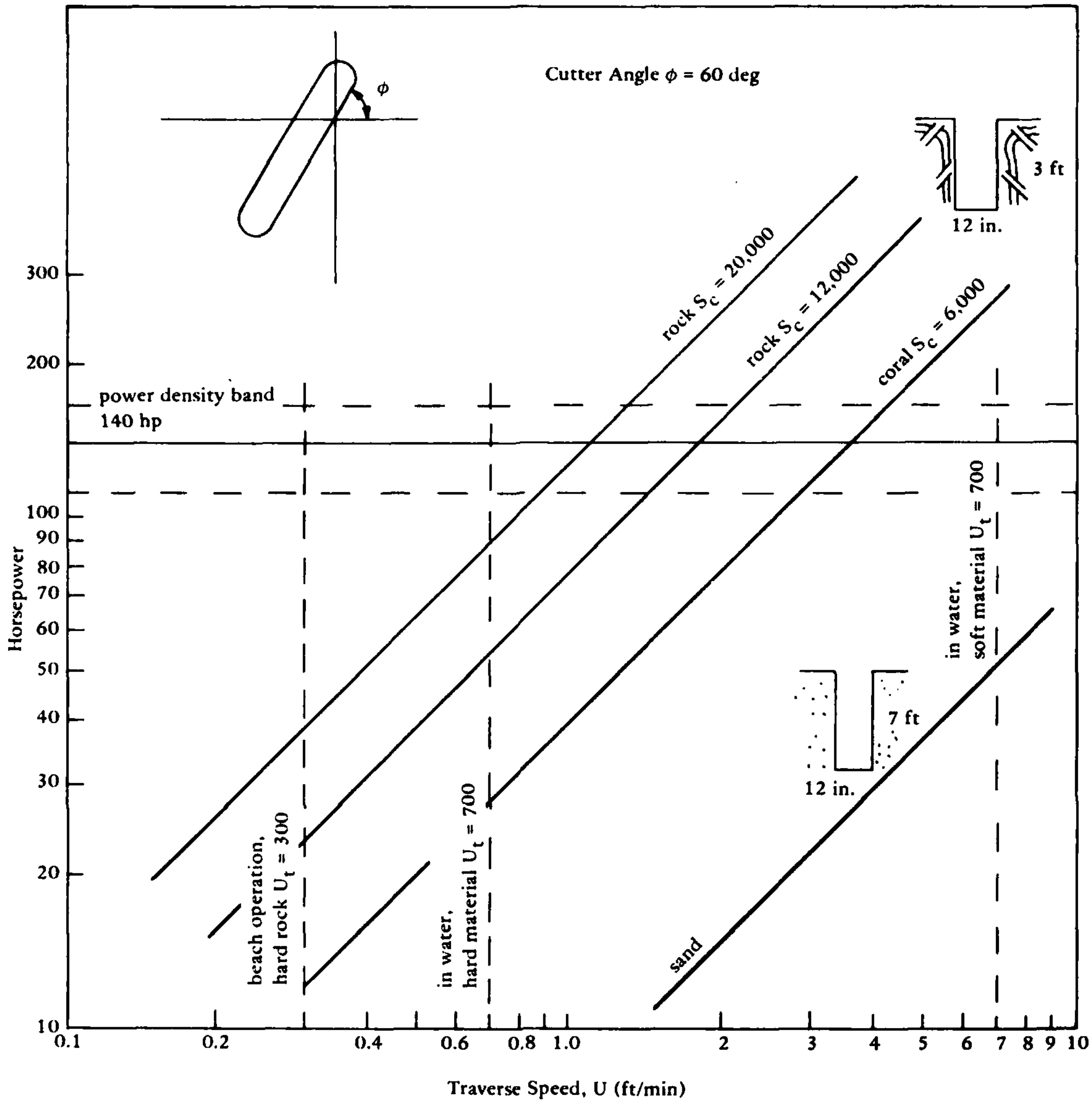


Figure 4-77. Theoretical horsepower requirements for 12-in.-wide trench.

tooth wear and breakage may limit the advance rate to between 0.3 and 0.7 ft/min for hard rock on the beach and underwater, respectively. A maximum advance rate of 7 ft/min may be obtainable for softer rocks underwater.

Selection Factors.

BOTTOM MATERIAL AND TOPOGRAPHY. Mechanical trenching is feasible in most nearshore bottom materials with the possible exception of very hard rock, such as basalt. The maximum trench depth of 7 feet proposed in the design concept would allow burial in sand to a depth sufficient to avoid most anchor drag problems and would also provide an adequate trench in rock with as much as 4 feet of sand cover. At sites where rock is buried more than 7 feet below the seafloor surface at the time of installation but exposed at other times of the year due to sand transport, this technique is of very little value.

Since mechanical trenchers are bottom-crawling vehicles, topography plays a major role in determining the feasibility of operating at a specific site. The modified Vermeer trencher could only negotiate obstacles <6 inches high, while the proposed concept would allow obstacle negotiation of 2 feet. Sites with very irregular topography will either make this technique infeasible or require time-consuming and costly route preparation.

WAVES AND CURRENTS. Once the trencher is through the surfzone, waves and current will have negligible effect on the trenching machine during remote-control operations. If it is to be operated by divers, then near-calm conditions must exist. Current or surge >1 knot will make diver control of the trencher difficult or dangerous. A computer analysis of various crawler vehicle designs subjected to hydrodynamic wave loads is currently being conducted by the Army's Waterways Experiment Station (WES), to determine surfzone translation capabilities of the different configurations.

LOGISTICS. The weight and size of the trencher system, support equipment, and consumables may make this technique infeasible at remote sites. The diesel electric generator requires 220 gallons of diesel fuel for every 8 hours of operation, and the weight of the trencher system components is estimated as over 70 tons. Because of the sophistication of several of the subsystems, a field maintenance and repair facility must be provided at the site or shipped with the trencher.

WEATHER WINDOW. The weather window must be long enough to allow trenching through the surfzone and out to a depth where hydrodynamic forces from large waves will not affect operation of the trencher. Once beyond this point, remote control trenching operation can proceed regardless of sea conditions. Estimates of this critical depth versus wave height are not currently available but are anticipated as a result of the study being conducted by WES. Sea conditions must also be favorable when returning the trencher to the beach at the conclusion of the stabilization operation.

VISIBILITY. Good underwater visibility (>10 feet) is required for route survey and preparation and for underwater maintenance of the trencher. Visibility during remote control operations is not as critical; and, unless special considerations are taken to remove the trench spoil from the area, visibility in the vicinity of the trencher during its operation is likely to be near zero.

HAZARDS. Installing a cable in a narrow trench below the seafloor surface is one of the best methods of protecting it from hydrodynamic forces and the hazards discussed in Chapter 2.

WIND. Wind has no effect on remote operation of the trencher. When diving operations are required for either route survey or underwater maintenance of the trencher, the expected wind velocity should not exceed 20 knots.

DESIGN LIFE. This technique can provide protection and stabilization of cables for periods in excess of 20 years. Because of the cost of deploying the required hardware and personnel, trenching is probably not cost-effective for short duration installations (<5-year life requirements).

LENGTH OF PROTECTED CABLE. Protection lengths greater than 2,500 feet will require a surface support platform onto which the power sources and control station can be transferred. An economic analysis has shown that trenching becomes cost-competitive with split-pipe stabilization for cable runs of 1,200 feet or more.

4.4.6 Drilled Hole

Background and Description. Cables can be installed in drill-holes starting just above the high water mark, traversing the nearshore at some distance below the seafloor, and emerging from the seabed in deep water beyond the more aggressive environments. In rare instances, the required drill-hole is straight and short and can thus be drilled by a light mobile rig of the type used in mining and construction exploration. The one known completed installation of this type is on the south coast of Greenland, constructed to prevent grounding icebergs from crushing the commercial telephone cables. The drill-holes are straight bores, 650 feet (200 meters) long, inclined at 36 degrees from the horizontal to emerge from the seafloor basalt 340 feet (103 meters) below sealevel (Pederson, 1974). More often, the required drill-hole is curved, starting downward at the shore end and then turning and angling back upward to emerge from the seabed some distance offshore (Figures 4-78 and 4-79). More sophisticated drilling techniques and equipment are required to complete such curved drill-holes. Boring machines, capable of starting at a very shallow angle, are limited by available thrust to traverse horizontal distances of 4,000 to 5,000 feet (Anon., 1974b; Katz, 1975). Large oil-field-type drill rigs, using weighting on the

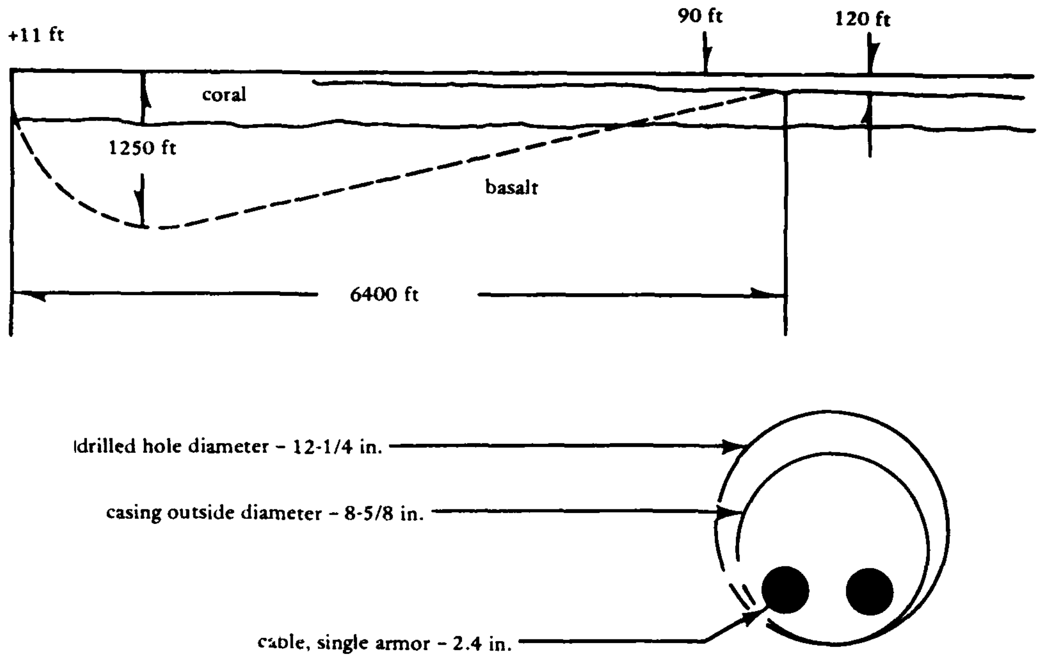


Figure 4-78. Oil rig drill-hole profile for 6,400-ft traverse in coral. Cross section shows drill-hole, casing, and cable relationship (Valent and Brackett, 1976).

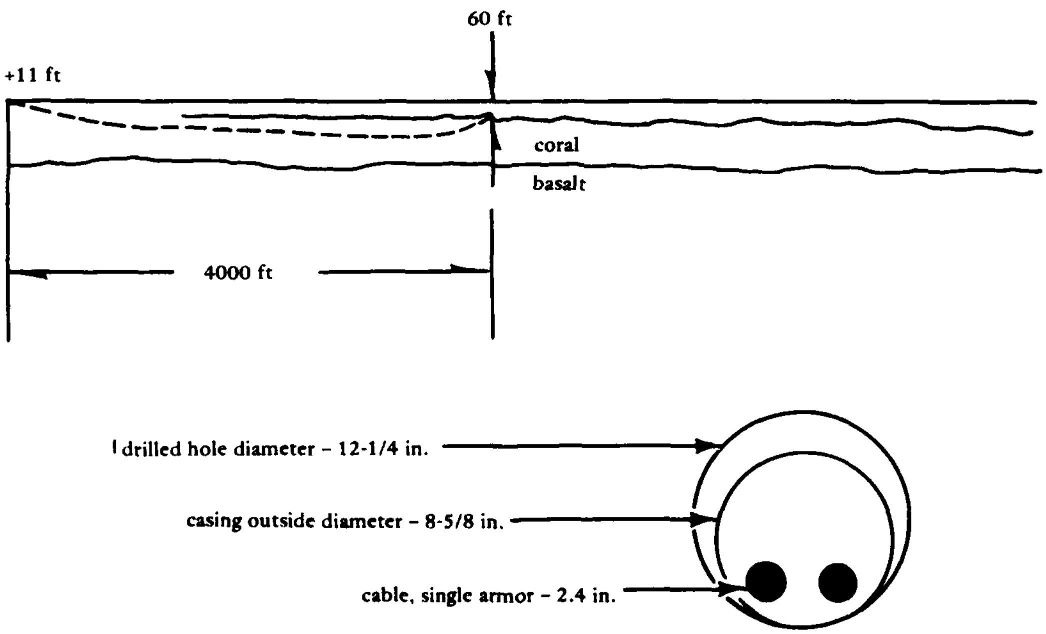


Figure 4-79. Horizontal pipeline boring rig drill-hole profile for 4,000-ft traverse. Cross section shows drill-hole, casing, and cable relationship (Valent and Brackett, 1977).

drill-string and rig thrust, are able to develop greater drill-bit thrust; Littlejohn (1974) indicates such rigs should be capable of a horizontal traverse of 13,000 feet. Thus far, the longest reported horizontal drilled hole is 1,685 feet, carrying a pipeline beneath a river (Emery, 1974).

The use of drilled holes to carry electrical communication cables beneath the nearshore appears economically justified only in very special circumstances; essentially, the required traverse to 60 ft of water must be no longer than 4,000 ft or if the seafloor material is softer rock or soil, then other techniques (e.g., a form of trenching) will prove more cost-effective (Valent and Brackett, 1976).

Even under the stated requirements, use of a drill-hole for a cable traverse may not be advisable because of geologic conditions not conducive to drilling or because of logistic problems in getting the bulky, heavy drill rigs to the site and in finding a suitable area to set up and spud the drill-hole.

Procedure. The exact sequence of operations in drilling and casing the drill-hole is complex, dependent on the geology, the rig type, and the drill-hole designer. Drilling is initiated,using a conventional bit and rotating drill-string to produce a straight section of hole into which a surface casing is set. Drilling then continues with additional casing strings of successively smaller diameter being installed as necessary to maintain the drill hole. Turning of the drill-hole direction is done by using a specially designed down-hole drill motor (Garrison, 1967). After egress of the drill-bit from the seafloor, the electric cable could be lowered from a surface platform, attached to the end of the drill-string, and pulled into the cased hole as the drill-strings are recovered at the rig. Alternately, a pulling line could be left in the cased hole so the electrical cable could be installed at some later date.

Support Requirements and Installation Time Estimates. Support requirements for the drill-hole completion are normally the entire responsibility of the drilling contractor. They include a wide variety of equipment and personnel for transport of rig, casing, drilling, mud, and cement; batching and pumping of mud and cement; and drill-hole survey and directional control. Installation time varies with the complexity of the job. Time estimates for a 13,000-foot drill-hole were 14 days for setup, 322 days for drilling and casing of the hole, and 8 days for demobilization (i.e., 1 year). The time estimate for a more modest 4,000-foot drill-hole was 120 days total (i.e., 4 months).

Installation of the elctrical cable will require an anchored barge at the drill-hole mouth and a pulling winch at the land end. Divers will be needed to check conditions at the sea-end of the drill-hole.

Total costs, including cost of two cables in the one drill-hole, were estimated at $400/ft for the 13,000-foot traverse and $200/ft for the 4,000-foot traverse.

Selection Factors.

BOTTOM MATERIAL AND TOPOGRAPHY. The seafloor rock type, the local geology (e.g., faults, jointing, seams of sand or weathered material, voids), and the topography (particularly the length of traverse to reach a safe water depth) are each critical in determining whether a drill-hole installation is viable at a given site.

WAVES AND CURRENTS. Waves and currents have little direct impact on the success of a given drill-hole installation because the drill-hole passes under the worst zone. However, waves and currents do have indirect impact on the selection of a drill-hole installation by making more conventional approaches to cable installation less attractive.

LOGISTICS SUPPORT. Use of a drill-hole is not particularly sensitive to logistics problems because the actual drilling and casing operation can be carried out well in advance of the cable installation, and problems arising during the drilling and casing therefore should not impact on the cable installation. A drill-hole installation may offer significant advantages over the "across-the-surface" options because access is not required to the beach and surf areas.

HAZARDS. Within the length of the drill-hole, the cable installation is immune to all stated hazards. Accommodation can even be made for the potential pinching-off of the casing in active fault zones by insertion of reamed sections of hole in the zones of suspected movement.

DESIGN LIFE. The drill-hole option is a long-lived option; it is far too expensive to be entertained for short-lived installations (e.g., <5 years).

4.5 TENSIONING

4.5.1 Background and Description

Tensioning of a cable provides a degree of stabilization by limiting the movement of a cable. The nearshore cable is anchored at either the shore end or at the sea end while it is still supported at the sea surface by the installation floats. Then the cable is tensioned at the other end and that tension maintained while the floats are removed and the cable is placed on the seafloor. A mushroom or clump anchor is installed to maintain the tension. This technique is known to have worked quite well with a single-armored cable in a nonaggressive environment on moderately hard, exposed limestone with some sand cover.

Although tensioning has rarely been used as the only means of protecting an ocean cable, it is presented as a separate technique since it can be used with almost any of the other cable protection techniques to reduce the magnitude of the displacement produced by hydrodynamic forces. The effects of tensioning on the system design are discussed in detail in Chapter 6.

Some degree of tensioning during installation is essential to eliminate unnecessary slack and excess cable use and to minimize the potential for tangling and hockling. However, in certain environments the maintenance of high tension after installation will not be sufficient to protect the cable without the addition of other immobilization techniques. Generally, on sand, sufficient slack should be left in a submarine cable such that it will conform with the changing seafloor topography.

4.5.2 Procedure

Tensioning of a cable is accomplished during laying of the cable on the seafloor. The shore end of the cable is tied to a sufficient deadweight anchor, on or just behind the beach, and the sea end is tied to a mushroom or deadweight anchor in order to maintain tension.

4.5.3 Support Requirements and Installation Time Estimates

Support requirements include a work barge or boat capable of installing the required sea-end anchor and capable of applying the required tension force at the sea end if installed with the sea end last. On the shore end, tension can be applied by a crawler tractor if sufficient "runway" length is available or by a large winch anchored near the deadweight anchor. No specialized personnel are required for this task. Perhaps a half-day extra ship time is required for installation of the sea-end anchor, and 2 days' time for four men to prepare a suitable deadweight anchor on-shore.

4.5.4 Selection Factors

Bottom Material and Topography. Cable tensioning as-installed has been used effectively to immobilize cables laying exposed on soft to medium-hard seafloor rock. However, tensioning is not expected to prove satisfactory on exposed, hard rock seafloors (basalt) where the rock is considerably harder than the steel of the armor wires. This is especially true where bottom discontinuities 3 to 6 feet in height promote cable suspensions and points of aggravated abrasion. On such hard-rock seafloor, additional immobilization or protection of the cable from wave and current forces is necessary. Even cables on seafloors of exposed softer rock (e.g., coral) are not commonly safe when immobilized by tension alone; additional weighting and even tie-downs are usually necessary. Thus, it is very important when considering tensioning as an immobilization technique to assess the seafloor topography and material type in order to determine if tensioning is suitable and adequate.

Waves and Currents. The magnitudes of the wave and current forces applied to the cable and of the attached marine growth are necessary to determine whether or not the cable will be moved by such forces and made to abraid itself. Equations for predicting these forces on cables lying on the seafloor are presented in Chapter 5

Logistics Support. Logistics for this technique have been addressed in Section 4.3.3. In addition to those comments it should be said that tensioning does not necessarily require good access down to the water's edge or even near the beach area, provided the cable can be tensioned along a near-straight line.

Weather Window. Installation of the cable and tensioning against the anchors must be accomplished in one weather window. Installation of the sea-end anchor and tensioning of the cable should require 2 days' extra time. At most, this minimal time extension should not normally compromise an installation operation.

Hazards. Tensioning of a cable can potentially aggravate hazard problems (see Section 2.4). Tensioning can limit self-burial of a cable due to wave action and may even result in suspension of a cable above the seafloor, making it more vulnerable to hooking and damage by a dragging anchor. Tensioning can also aggravate scouring problems by preventing the cable from conforming to developing seafloor profiles and by increasing the possible length of cable suspensions spanning scour depressions. Tensioning should have little impact on the vulnerability of a cable system to other natural hazards. Therefore, only the potential for dragging anchor incidents and for scour need be thoroughly assessed when evaluating the use of tensioning for cable stabilization.

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FORCES ACTING ON OCEAN CABLES

5.1 INTRODUCTION

Ocean cables are subjected to both static and dynamic forces as a result of their presence in the ocean environment. Two basic types of static loads affect the cable: (1) forces produced by the reaction of the weight of the cable resting on or suspended above the seafloor, and (2) hydrostatic pressure. Dynamic loads consist of forces produced by currents and waves as water particles move past the cable. This discussion will not treat the forces produced by dragging anchors, trawling gear, or grounding icebergs since they do not lend themselves to generalized analytical solutions.

The configuration of the cable system will also have an impact on both the type and magnitude of the forces exerted on the cable. Two basic configurations are encountered with ocean cables: (1) bottom resting, and (2) horizontally suspended cables. In reality both of these configurations are usually encountered at different points along the length of a single cable. The cross-sectional configuration of the cable/stabilization system also influences the magnitude of the dynamic forces exerted on the cable.

Water particle motion caused by currents and waves produces the dynamic forces of interest. The motion produced by currents will be considered steady state for the purpose of this discussion, since it generally changes slowly with respect to time. Wave-induced motion on the other hand is oscillatory in nature and, thus, both the velocity and

acceleration of the water particles change rapidly, going through one complete cycle during the period of one wave. In the nearshore region, the wave-induced water particle motion follows an elliptical orbit; however, in the area of interest near the seafloor, the vertical components of both velocity and acceleration become very small, and the motion of the water particles can be considered to be rectilinear.

Analytical determination of the forces exerted on ocean cables requires certain simplifying assumptions to be made that may result in a divergence between the modeled environment and that actually defined as a result of the site survey. The assumptions required for development of the analytical solution are presented in Section 5.4.

5.2 STATIC LOADS

5.2.1 Seafloor Reaction Forces

In the absence of dynamic forces, the reaction force produced by the seafloor on a bottom-resting cable is equal in magnitude and opposite in direction to the submerged weight of the cable plus the weight of the stabilization system components (in the case of mass anchors) or vertical clamping load (in the case of tie-down systems). The submerged weight of cables and stabilization systems can usually be obtained either from the manufacturer, from Chapter 4 of this handbook, or calculated from the following equation:

$$W_s^* = \sum \rho_i V_i^* - \rho_w \sum V_i^* \qquad (5\text{-}1)$$

where W_s^* = submerged weight per unit length
ρ_i = density of i^{th} component of cable/stabilization system
V_i^* = volume per unit length of i^{th} component
ρ_w = density of seawater

Table 5-1 provides the submerged unit weight of some of the most common cables and stabilization system components.

Table 5-1. Weight and Density of Typical Cable Stabilization System Components

Component	Weight Per Unit Length		Density, ρ_i (lb/ft^3)
	In Air, W* (lb/ft)	In Water, W_s* (lb/ft)	
Cables			
SDC List 3	5.27	3.56	194
SDC List 4	7.28	5.27	226
SDC List 5	14.75	11.45	280
Stabilization System Components			
Concrete	-	-	160
Split-Pipe			
3-1/2-in. ID	43	40	450
5-in. ID	60.4	57.2	450
Chain (stud link)			
2 in.	39.2	34	485
2-1/2 in.	61.4	53.3	485
3 in.	89.3	77.5	485
Chain (close link)			
2 in.	40	34.7	485
2-1/2 in.	65	56.4	485
3 in.	86	74.6	485

5.2.2 Hydrostatic Loads

In general the vertical dimensions of the cable and stabilization system are very small compared to the depth of water of interest and, therefore, the hydrostatic pressure can be considered unifrom and equal to the pressure existing at a depth equivalent to the distance between the still water line and the center of the cable. Hydrostatic pressure increases at a rate of approximately 0.445 psi per foot of water depth, but this does vary slightly as a function of salinity and temperature.

Most ocean cables are designed to withstand the hydrostatic pressures that exist even in the deep ocean. Hydrostatic pressure becomes a problem only when some defect exists (either from fabrication or damage caused during installation) that allows the pressure to force water into contact with one of the conductors, thereby causing an electrical fault.

5.2.3 Loads Due to Cable Suspensions

Cable suspensions are most common on irregular rock or coral seafloors. However, they can occur on seafloors that experience large

annual topographic changes if the cable is laid with too much tension. On rocky seafloors, suspensions up to 60 feet in length are common, while a few isolated cases of suspensions exceeding 250 feet have been reported (Cullison, 1975).

Because of the bending stiffness of nearshore ocean cables, short suspensions resemble a rigidly supported indeterminate beam that are difficult to analyze; however, for suspensions greater than about 40 feet, the portion of the load supported by end point bending moments becomes negligible, and the forces can be calculated using standard flexible cable analysis. For suspensions with small sag-to-span ratios ($S/2\ell_s < 0.1$), the minimum tension in the cable can be approximated from the parabolic function

$$T_o = \frac{W_s^* \ell_s^2}{2 S} \tag{5-2}$$

where W_s^* = submerged weight per unit length of cable

S = maximum cable sag

ℓ_s = horizontal distance from low point of cable to support point

T_o = minimum cable tension (at low point of cable)

The maximum tension occurs at the support points and is equal to:

$$T_{max} = W_s^* \ell \left(\frac{\ell_s^2}{4 S^2} + 1 \right)^{1/2} \tag{5-3}$$

For cables with sag-to-span ratios greater than 0.1, the unit weight cannot be assumed to be evenly distributed across the span of the cable, and the internal forces must be calculated from the following:

$$T_o = W_s^* c \qquad (5\text{-}4)$$

and

$$T_{max} = W_s^* c \cosh \frac{\ell_s}{c} \qquad (5\text{-}5)$$

The term c (the parameter of the catenary) is obtained from the equation:

$$c = \frac{\ell_c^{\,2} - S^2}{2\,S} \qquad (5\text{-}6)$$

where W_s^* = weight per unit length of cable

ℓ_s = horizontal distance from low point of suspension to support point

ℓ_c = length of cable from low point of suspension to support point

S = sag of cable

Figure 5-1 defines the coordinate system. When calculating the reaction forces of suspended cables, it is often more convenient to consider the x and z components of the force at the end points. For the parabolic analysis,

$$F_x = T_o = \frac{W_s^* \ell_s^{\,2}}{2\,S} \qquad (5\text{-}7)$$

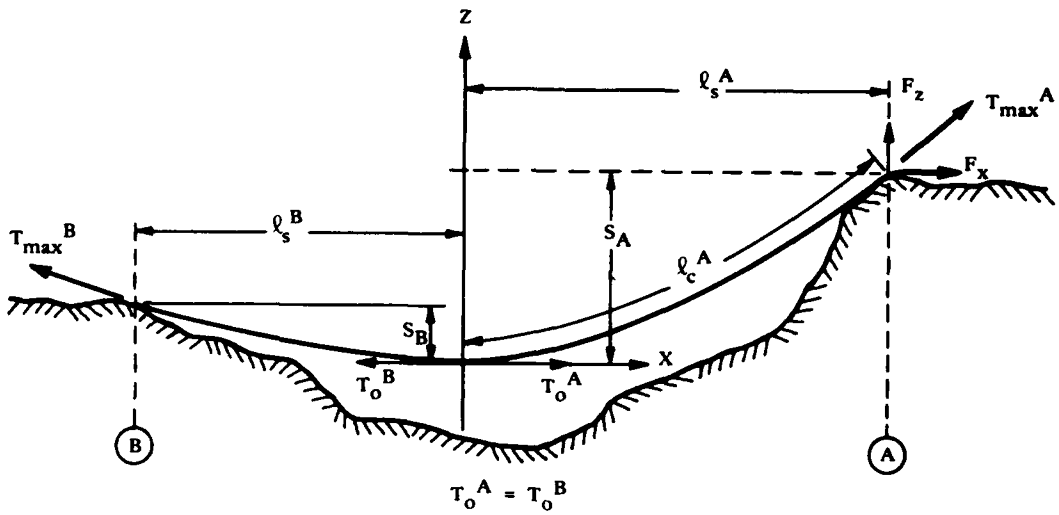


Figure 5-1. Coordinate system for cable suspension.

and

$$F_z = W_s^* \ell_s \tag{5-8}$$

For the catenary analysis,

$$F_x = T_o = W_s^* \left(\frac{\ell_c^2 - S^2}{2\,S} \right) \tag{5-9}$$

and

$$F_z = W_s^* \ell_c \tag{5-10}$$

Since cable installations are not usually planned to have large suspensions, the preceding discussion will not generally be used for preinstallation design. However, the installed configuration of cables

on irregular seafloors must be investigated to assure that the existence of large, suspended sections will not adversely affect the installation of the stabilization system, and that the stabilization of the end points is sufficient to withstand both static and dynamic loads on the suspended section of cable.

5.3 DYNAMIC LOADS

Forces exerted on cables due to the motion of water particles are divided into two general categories: (1) those due to steady state flow, and (2) those due to accelerated flow. Steady state flow is one in which the water particle velocity moving past a point does not change during the finite time period of interest. This time period may be as long as a day for major ocean currents or as short as a few seconds for wave-induced motion. Since the reaction of cable systems to hydrodynamic disturbances can be considered to be instantaneous, the time period of interest for maximum design conditions may be assumed to be sufficiently small to include current velocities and maximum instantaneous wave-induced surge velocities as "steady state" conditions.

Accelerated flow will, for the purposes of this discussion, only consider the change in velocity with respect to time produced by wave-induced particle motion. Even though tidal currents often do exhibit measurable acceleration components, they are on the order of several magnitudes smaller than accelerations produced by wave motion and can be neglected without affecting the results of the analysis. As previously mentioned, in the region near the seafloor the vertical components of velocity and acceleration are very nearly zero and can be neglected; therefore, accelerated flow-induced forces will be considered to act only in the horizontal plane.

5.3.1 Steady State Flow Forces

Forces resulting from constant velocity flow can be separated into two components: one acting in the horizontal plane, called the drag force, and the other acting in the vertical plane, referred to as the lift force. The magnitude of these forces are given by the following equation:

$$F_D = \frac{1}{2} C_D \rho A u^2 \tag{5-11}$$

and

$$F_L = \frac{1}{2} C_L \rho A u^2 \tag{5-12}$$

where F_D = horizontal (drag) force acting on the object (lb)

F_L = vertical (lift) force acting on the object (lb)

C_D = dimensionless drag coefficient

C_L = dimensionless lift coefficient

ρ = density of the fluid medium ($lb\text{-}sec^2/ft^4$)

A = projected area of the object perpendicular to the flow path (ft^2)

u = free stream velocity of the fluid (ft/sec)

The drag force is produced by two separate phenomena: form drag, which results from the pressure differential between the upstream and downstream sides of the object, and skin drag, which results from the shear stress between the surface of the object and the moving fluid. Both of these are a function of the Reynolds Number (R_e). For wave and current velocities of interest, R_e is on the order of 10^5, which results in the skin drag contributing only 1 to 2% of the total

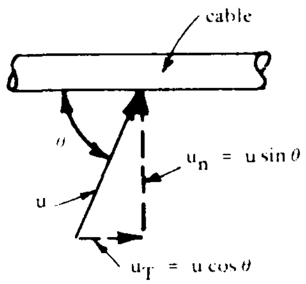


Figure 5-2. Vector components of velocity field.

drag force (Cullison, 1975). For the purpose of this analysis, the drag force can be considered to be composed of only form drag.

Both the coefficients of lift and drag are empirically derived dimensionless numbers that are a function of the shape of the object, Reynolds Number, height of the object off the bottom, and angle between the object and the direction in which the fluid is moving. The effect of these parameters on the lift and drag coefficients is discussed in detail in Section 5.5. The values of these coefficients, which have been corrected for the effect of the various parameters, will be referred to as the combined coefficients of lift and drag.

Since skin friction along the length of the cable has very little effect on the stability of the cable for the conditions encountered in the nearshore zone, only the velocity component acting perpendicular to the cable path is considered for force calculations. The magnitude of the force may be calculated using two methods:

(1) The vector component of the velocity acting perpendicular to the cable path may be calculated from the trigonometric relationship $u_n = u \sin \theta$ as shown in Figure 5-2.

(2) The coefficients of lift and drag may be modified to account for the angle of incidence between the velocity field and the cable path. Coefficient modification curves for both lift and drag were developed by Grace (1971) from empirical data to account for the effect of the angle of incidence. These curves will be discussed in more detail in Section 5.5.

The direction of the induced force, regardless of the method of calculation, is perpendicular to the cable path, with the drag force acting horizontally and the lift force vertically. Since the coefficient modification curves were developed from experimental data and give a more conservative design solution for angles greater than 30 degrees, this technique will be used throughout the remainder of the discussion.

5.3.2 Accelerated Flow Forces

Objects submerged in an accelerating flow field are subjected to forces resulting from the instantaneous velocity of the fluid (as discussed in the previous section), plus an added force required to cause the accelerating particles to move around the object. This force is referred to as the inertia force and can be expressed by the equation:

$$F_I = C_I \rho V \frac{du}{dt} \tag{5-13}$$

where F_I = inertia force (lb)

C_I = dimensionless coefficient of inertia

ρ = density of the fluid ($lb\text{-}sec^2/ft^4$)

V = volume of fluid displaced by the object (ft^3)

du/dt = acceleration of the fluid (ft/sec^2)

Since vertical accelerations near the seafloor approach zero, it can be assumed that the inertia force acts only in the horizontal direction. Morrison et al. (1953) proposed that the total horizontal force (F_H) is the algebraic sum of the inertia and drag forces such that:

$$F_H = F_I + F_D = C_I \rho V \frac{du}{dt} + \frac{1}{2} C_D \rho A u^2 \tag{5-14}$$

There is some question as to whether this procedure is accurate (Wiegel, 1964); however, since it results in a conservative design

solution, it has generally been accepted for engineering applications. For typical design waves in the nearshore zone, the velocity terms are generally two orders of magnitude greater than the acceleration terms, and the inertia force may be neglected in the analysis of cable loads (Cullison, 1975). Some mass anchor stabilization systems may have extremely large volume-to-projected-area ratios, and, thusly, the inertia forces may be found to be significant for the total system design. When calculating the wave forces on the stabilization system, it is recommended, therefore, that both the drag and inertia forces be calculated for at least one point along the cable path to assure that the inertia force does not significantly contribute to the total load.

5.3.3 Unit Forces

Cable systems exposed to hydrodynamic loads in the nearshore zone are extremely long, often approaching several miles in length. The hydrodynamic loads acting over such a length are usually not uniform in either magnitude or direction. Therefore, the forces are most often calculated for a unit length of cable at various locations along the cable path. If F* is designated as the force per unit length, then Equations 5-11, 5-12, and 5-13 become:

$$F_D^* = \frac{1}{2} C_D \rho D u^2 \qquad (5\text{-}15)$$

$$F_L^* = \frac{1}{2} C_L \rho D u^2 \qquad (5\text{-}16)$$

$$F_I^* = \frac{\pi}{4} C_I \rho D^2 \frac{du}{dt} \qquad (5\text{-}17)$$

where $F^* = \frac{F}{L} = \frac{\text{force}}{\text{unit length}}$

D = diameter of the cable (note: $A = D \cdot 1 = D$)

5.4 KINEMATICS OF WAVE MOTION

5.4.1 Background

Hydrodynamic forces exerted on the cables and their associated stabilization systems are a result of the motion of water particles produced by currents and waves. The motion produced by currents at a point of interest along a cable is relatively uniform, steady, and easily predicted or directly measured. Particle motion produced by waves, on the other hand, is not so easily obtained. There are at least twelve wave theories that have been used to calculate water particle velocities and accelerations. Each of these theories has shown good correlation to experimental data in different regions and for various wave parameters. No single theory, however, has been developed that adequately models the kinematics of wave motion from deep water through the surfzone and at every point in the water column.

Cullison (1975), citing the work of Goda (1964), LeMehaute et al. (1968), and others, has proposed the use of linear (Airy) wave theory for determination of particle velocities and accelerations near the seafloor. The selection of this theory is based on the fact that the solution of lineary theory for water particle motion is relatively simple and the correlation between experimental and theoretical results appears to be quite good near the seafloor. The correlation between theory and experimental results for water particle velocity was found to agree to within 10 to 15% (Goda, 1964; Iwagaki et al., 1970), while calculated values of acceleration require a constant multiplier of 1.5 to match experimentally observed accelerations (Grace and Rocheleau, 1973). Since force coefficients (C_D, C_L, C_I) are derived by comparing theoretical and experimental results for identical conditions, it is important that the force calculation be made using the same theory from which the force coefficients were obtained. A study conducted by Davis and Ciani (1976) found that most investigations of wave forces on horizontal

cylinders near the seafloor have used linear (Airy) wave theory to predict wave particle kinematics.

As waves approach the breaking zone in shallow water, linear theory becomes less valid; there is some indication that conoidal or stream function theory provides a better model up to the point of the waves actually breaking. The equations for both of these models are quite complex, however, compared to linear theory, and usually require the use of charts, tables, or computer programs for the solution of water particle motion. None of the existing theories provide a realistic model of the water particle motion from the breaker line to the shore. Grace and Castiel (1975) noted, however, that measured forces on submerged pipelines by breaking waves are more than twice as great as the forces predicted by a nonbreaking wave which could exist at the same location. A discussion presented in the Army Shore Protection Manual, Vol II (Army CERC, 1973), proposes calculating the velocity and accelerations produced by a wave about to break at the point of interest and then apply a correction factor of 2.5 to the coefficient of drag when calculating the horizontal forces. No such information about correcting the lift force has been presented.

Based on the previous discussion it is felt that the use of linear wave theory with appropriate empirical correction factors will best suit the requirement of this design guide. The use of this procedure allows reasonable and, in most cases, conservative engineering design solutions to ocean cable dynamics problems and also allows on-site calculations to be made that would be extremely difficult with most nonlinear theories.

5.4.2 Linear Wave Theory

Linear wave theory is based on the following assumptions:

(1) The fluid is homogeneous and incompressible, and the forces due to surface tension are negligible.

(2) The flow is irrotational.

(3) The bottom is impermeable and horizontal.

(4) The wave amplitude is small compared to the wave length and water depth.

(5) The pressure is constant along the air/sea interface.

The sea surface profile assumed by this theory is sinusoidal and has a constant amplitude and period. The equation for a point on this surface is given by

$$\eta = \frac{H}{2} \cos\left(\frac{2\pi x}{L} - \frac{2\pi t}{T}\right) \qquad (5\text{-}18)$$

Figure 5-3 defines the various wave parameters and coordinate system.

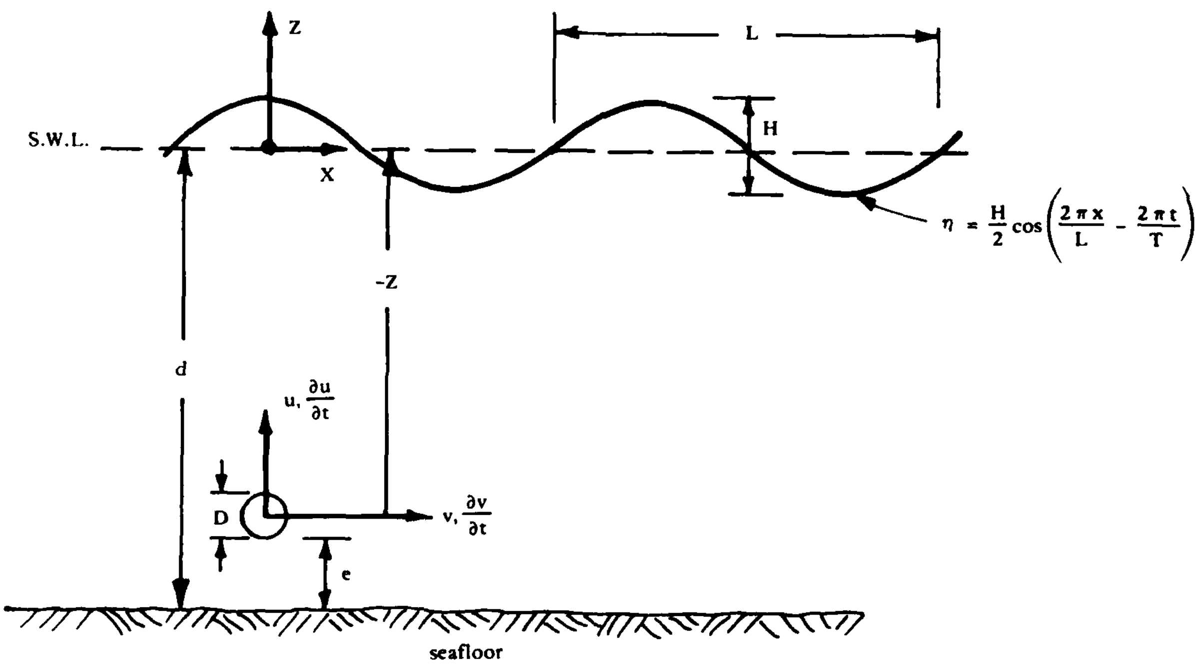


Figure 5-3. Wave parameters and coordinate system for linear wave theory.

The wave particle kinematics equations resulting from this theory are:

$$u = \frac{H\,\pi}{T}\left\{\frac{\cosh\left[\frac{2\,\pi}{L}\,(d + z)\right]}{\sinh\left(\frac{2\,\pi\,d}{L}\right)}\right\}\cos\left(\frac{2\,\pi\,x}{L} - \frac{2\,\pi\,t}{T}\right) \tag{5-19}$$

and

$$v = \frac{H\,\pi}{T}\left\{\frac{\sinh\left[\frac{2\,\pi}{L}\,(d + z)\right]}{\sinh\left(\frac{2\,\pi\,d}{L}\right)}\right\}\sin\left(\frac{2\,\pi\,x}{L} - \frac{2\,\pi\,t}{T}\right) \tag{5-20}$$

Therefore, the maximum velocities and accelerations are:

$$u_{max} = \frac{H\,\pi}{T}\left\{\frac{\cosh\left[\frac{2\,\pi}{L}\,(d + z)\right]}{\sinh\left(\frac{2\,\pi\,d}{L}\right)}\right\} \tag{5-21}$$

$$v_{max} = \frac{H\,\pi}{T}\left\{\frac{\sinh\left[\frac{2\,\pi}{L}\,(d + z)\right]}{\sinh\left(\frac{2\,\pi\,d}{L}\right)}\right\} \tag{5-22}$$

$$\left(\frac{\partial u}{\partial t}\right)_{max} = \dot{u}_{max} = \frac{2\,\pi^2\,H}{T^2}\left\{\frac{\cosh\left[\frac{2\,\pi}{L}\,(d + z)\right]}{\sinh\left(\frac{2\,\pi\,d}{L}\right)}\right\} \tag{5-23}$$

$$\left(\frac{\partial v}{\partial t}\right)_{max} = \dot{v}_{max} = \frac{2\,\pi^2\,H}{T^2}\left\{\frac{\sinh\left[\frac{2\,\pi}{L}\,(d + z)\right]}{\sinh\left(\frac{2\,\pi\,d}{L}\right)}\right\} \tag{5-24}$$

For cables either resting on or suspended very near the seafloor, $(d+z) \approx 0$, Equations 5-21 through 5-24 reduce to:

$$u_{max} = \frac{\pi H}{T \sinh\left(\frac{2 \pi d}{L}\right)} \tag{5-25}$$

$$\dot{u}_{max} = \frac{K_A \, 2 \pi^2 H}{T^2 \sinh\left(\frac{2 \pi d}{L}\right)} = \frac{3 \pi}{T} u_{max} \tag{5-26}$$

$$v_{max} = \dot{v}_{max} = 0 \tag{5-27}$$

where $K_A = 1.5$ (from Section 5.4.1)

The critical wave parameters that must be identified in order to calculate the water particle kinematics, therefore, are:

- H = wave height
- T = wave period
- L = wave length
- d = water depth
- The condition $d+z \approx 0$

If the last condition is not valid, then Equations 5-21 through 5-24 must be used to calculate the water particle kinematics.

As the wave moves from deep to shallow water, the wave period (T) remains constant, while the wave length (L) and height (H) vary with depth (d). This variation in wave characteristic parameters is due primarily to two phenomena: (1) shoaling, and (2) refraction.

5.4.3 Waves in Shoaling Water

The equations developed from linear theory were for waves in water of constant depth. As the wave approaches the shore and encounters the bottom where $d/L < 0.5$, the wave begins to "feel the presence" of the seafloor, and the wave height and wave length are altered. Following the approach developed by Rayleigh in 1911, it can be shown that in shoaling water:

$$\frac{H}{H_o'} = \left\{ \tanh \frac{2 \pi d}{L} \left[1 + \frac{4 \pi d/L}{\sinh(4 \pi d/L)} \right] \right\}^{-1/2} \tag{5-28}$$

and

$$\frac{L}{L_o} = \tanh \frac{2 \pi d}{L} \tag{5-29}$$

where H_o' = deep water wave height unaffected by refraction
L_o = deep water wave length
H = wave height at depth d
L = wave length at depth d

Since the shoaled wave length (L) is unknown and appears in both sides of Equation 5-29, the solutions to these equations are most easily obtained from graphs or tables. Figure 5-4 plots the values of H/H_o' and L/L_o as a function of d/L_o. Wiegel (1964) also presents tabulated values for these ratios as a function of d/L_o, which are reproduced in Appendix C.

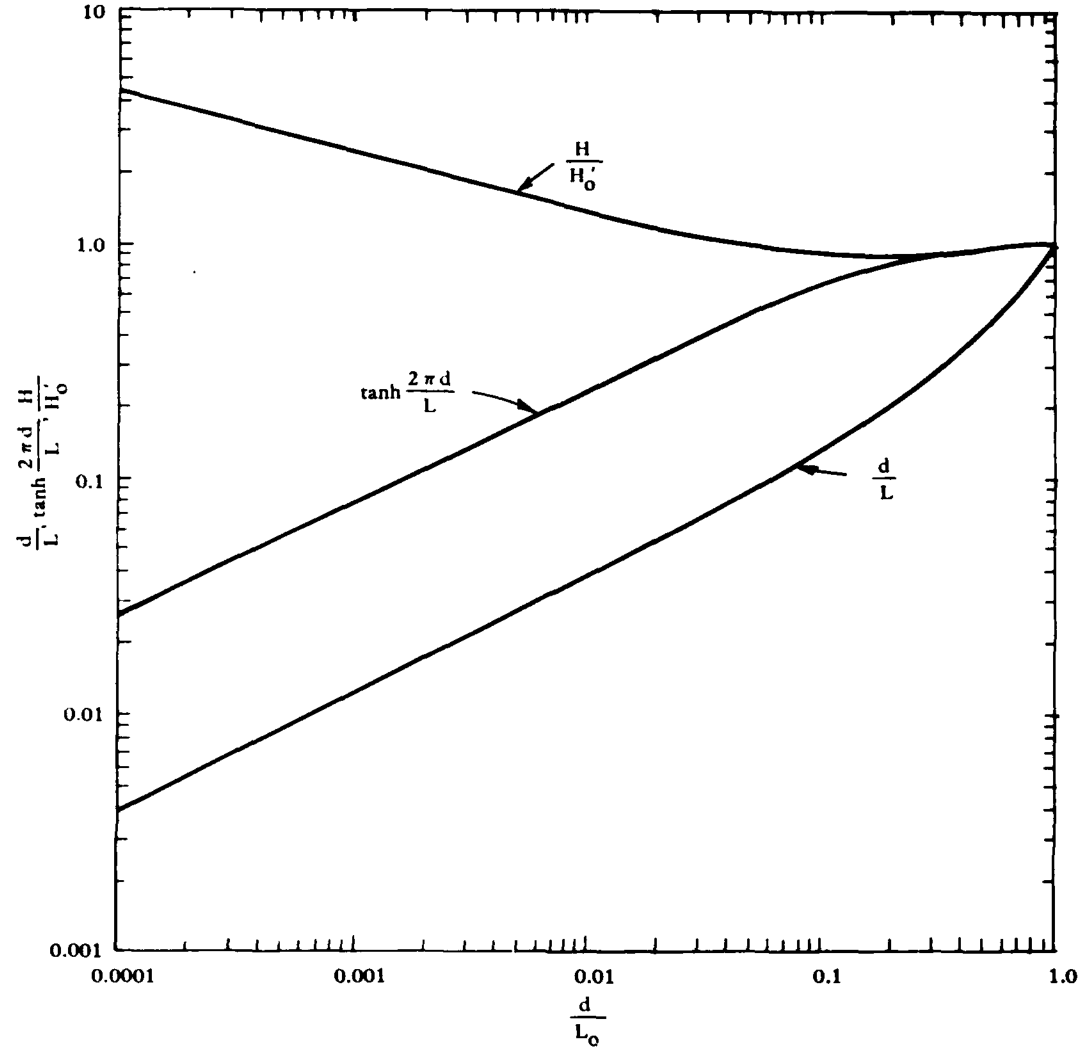


Figure 5-4. Value of various parameters as a function of d/L_O.

5.4.4 Wave Refraction

When waves reach the transitional water depth ($d/L < 0.5$), the phase velocity is no longer a function of the wave length alone, but becomes dependent on the depth of water as well. If the water depth is not constant along the crest of a wave, it will bend. This bending is known as wave refraction and, for linear wave theory, obeys Snell's law such that:

$$\frac{\sin \alpha}{\sin \alpha_o} = \frac{C}{C_o} \tag{5-30}$$

where α = angle between the wave front and the seafloor contour at the point of interest

α_o = angle between the deep water wave front and the contour

C = wave speed at the point of interest

C_o = speed of the deep water wave

This equation is based on the assumption that the seafloor contours are straight and parallel. Substituting for C/C_o, the angle between the wave crest and the contour is obtained from:

$$\alpha = \sin^{-1} \left(\tanh \frac{2 \pi d}{L} \sin \alpha_o \right) \tag{5-31}$$

where the term $\tanh(2\pi d/L)$ is evaluated from Appendix C or Figure 5-4, corresponding to the dimensionless depth parameter (d/L_o) for the point of interest.

Refraction also affects the height of a wave. Assuming that energy does not flow laterally along the wave crest, the height will be proportional to the length of the crest between orthogonals. Based on this assumption, the refracted wave height is obtained from the equation:

$$\frac{H_R}{H_o} = \left(\frac{b_o}{b_R} \right)^{1/2} \tag{5-32}$$

where H_R = height of refracted wave

H_o = height of deep water wave

b_R = crest length of refracted wave

b_o = crest length of deep water wave

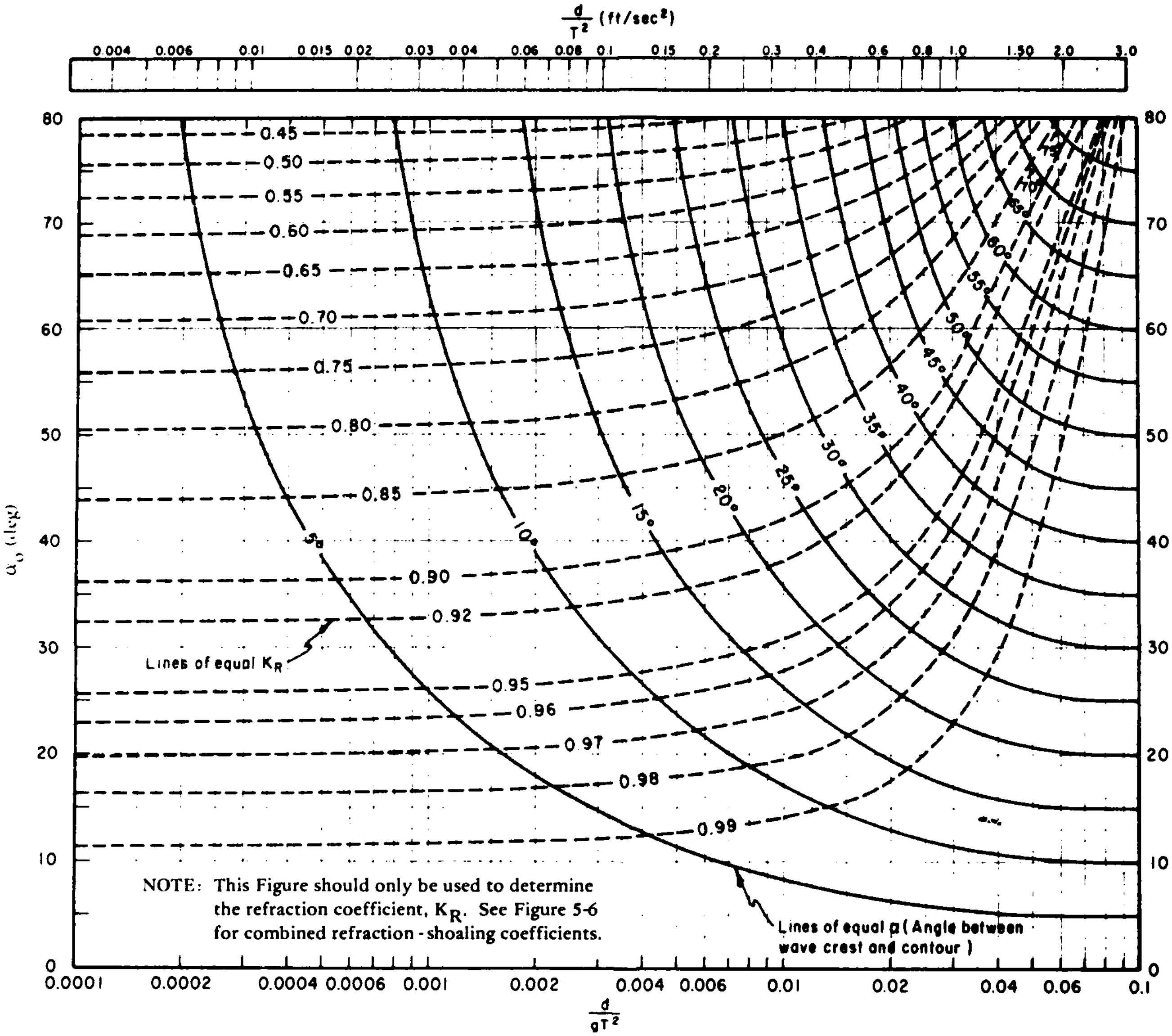


Figure 5-5. Changes in wave direction and height due to refraction on slopes with straight, parallel depth contours (from Shore Protection Manual).

The term $(b_o/b_R)^{1/2}$ is known as the refraction coefficient and is usually designated by K_R. The value of K_R may be computed from the relationship

$$K_R = \left(\frac{b_o}{b_R}\right)^{1/2} = \left(\frac{\cos \alpha_o}{\cos \alpha}\right)^{1/2} \tag{5-33}$$

To simplify the analysis of refracted waves, Figure 5-5 presents the relationship between α, α_o, K_R and the dimensionless depth parameter $d/(gT^2)$.

When a wave train approaches shallow water at an angle to the seafloor contours, it will be influenced by the combined effects of both shoaling and refraction. The height of a wave experiencing both refraction and shoaling is given by

$$H = H_o \left(\frac{H}{H_o'}\right) K_R \tag{5-34}$$

It will approach the beach at an angle α as given by Equation 5-31. Figure 5-6 presents in graph form the combined effect of refraction and shoaling on slopes with straight, parallel depth contours. In this figure, the coefficient $K_s = H/H_o'$ and from Equation 5-34:

$$\frac{H}{H_o} = K_R K_s \tag{5-35}$$

5.4.5 Construction of Refraction Diagrams by Orthogonal Method

The preceding discussion of refraction is based on a nearshore zone seafloor that can be modeled with straight and parallel contours.

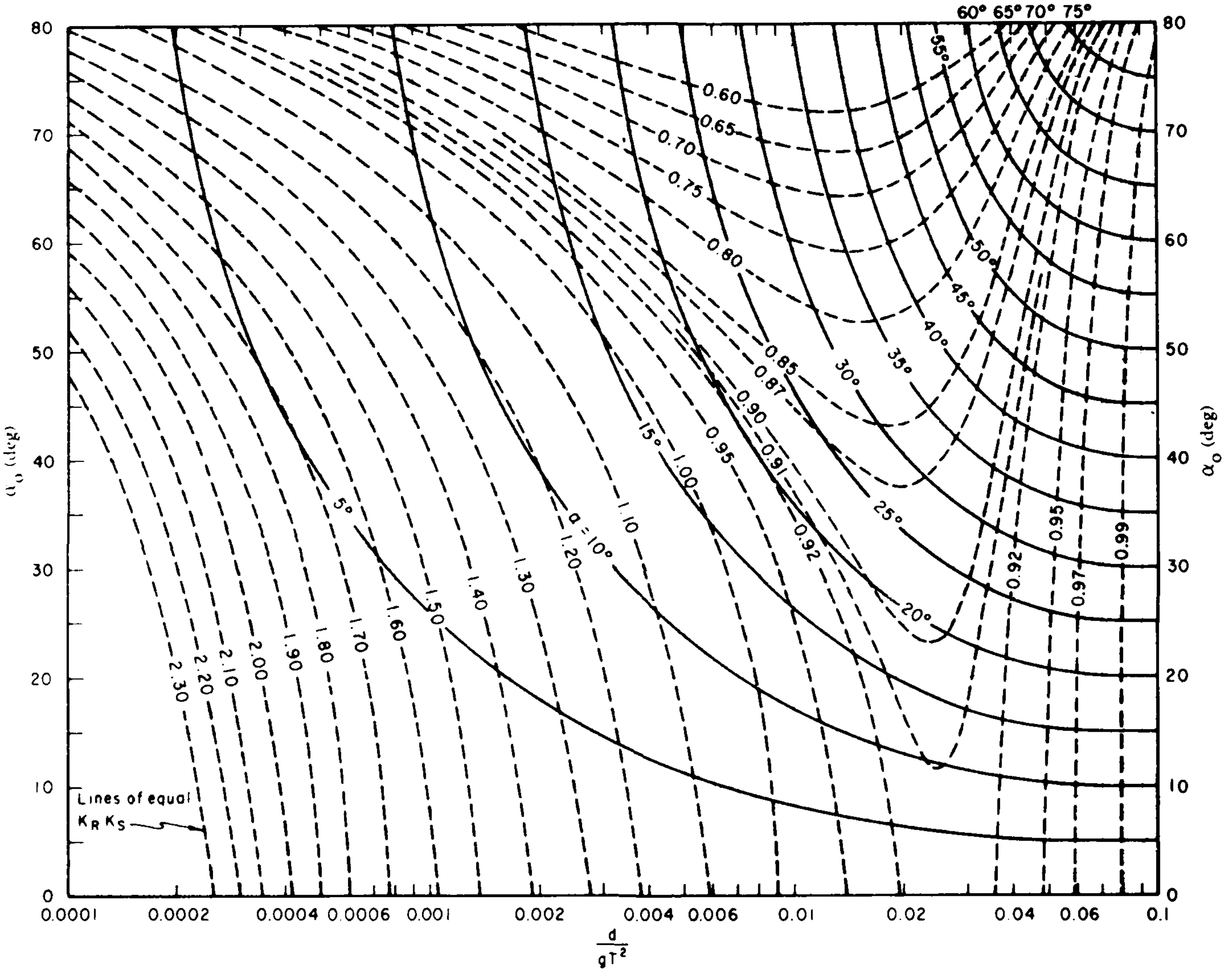


Figure 5-6. Change in wave direction and height due to refraction on slopes with straight, parallel depth contours including shoaling (from Shore Protection Manual).

When the actual conditions differ significantly from this idealistic model, such as bays, inlets, points of land, or extremely irregular seafloor contours, a graphical technique of refraction analysis is normally used. The technique of constructing refraction diagrams by the orthogonal method is discussed in detail by Wiegel (1964) and the U.S. Army Shore Protection Manual, Vol I (Army CERC, 1973), and is summarized here.

This technique utilizes a template, Figure 5-7, constructed from the relationship of α, α_2, C_1, and C_2 given by Snell's law and a topographic chart of the seafloor in the area of the cable installation. This procedure is based on the assumptions that:

(1) α_o is less than 80 degrees

(2) The slope of the seafloor is constant between contour lines

(3) The radius of curvature of the orthogonals between contours is constant (i.e., a circular arc)

The procedure for constructing an orthogonal refraction diagram is as follows:

(1) From a chart of the area or the results of the site survey, construct a topographic map of the seafloor from the shoreline out to a depth (d) such that d/L_o = 0.5. The selection of contour intervals and spacing between orthogonals will determine the accuracy of the results. (Accuracy increases as the spacing between contours and orthogonals decreases.)

(2) Construct orthogonals for the deep water wave up to the point where they meet the deepest contour.

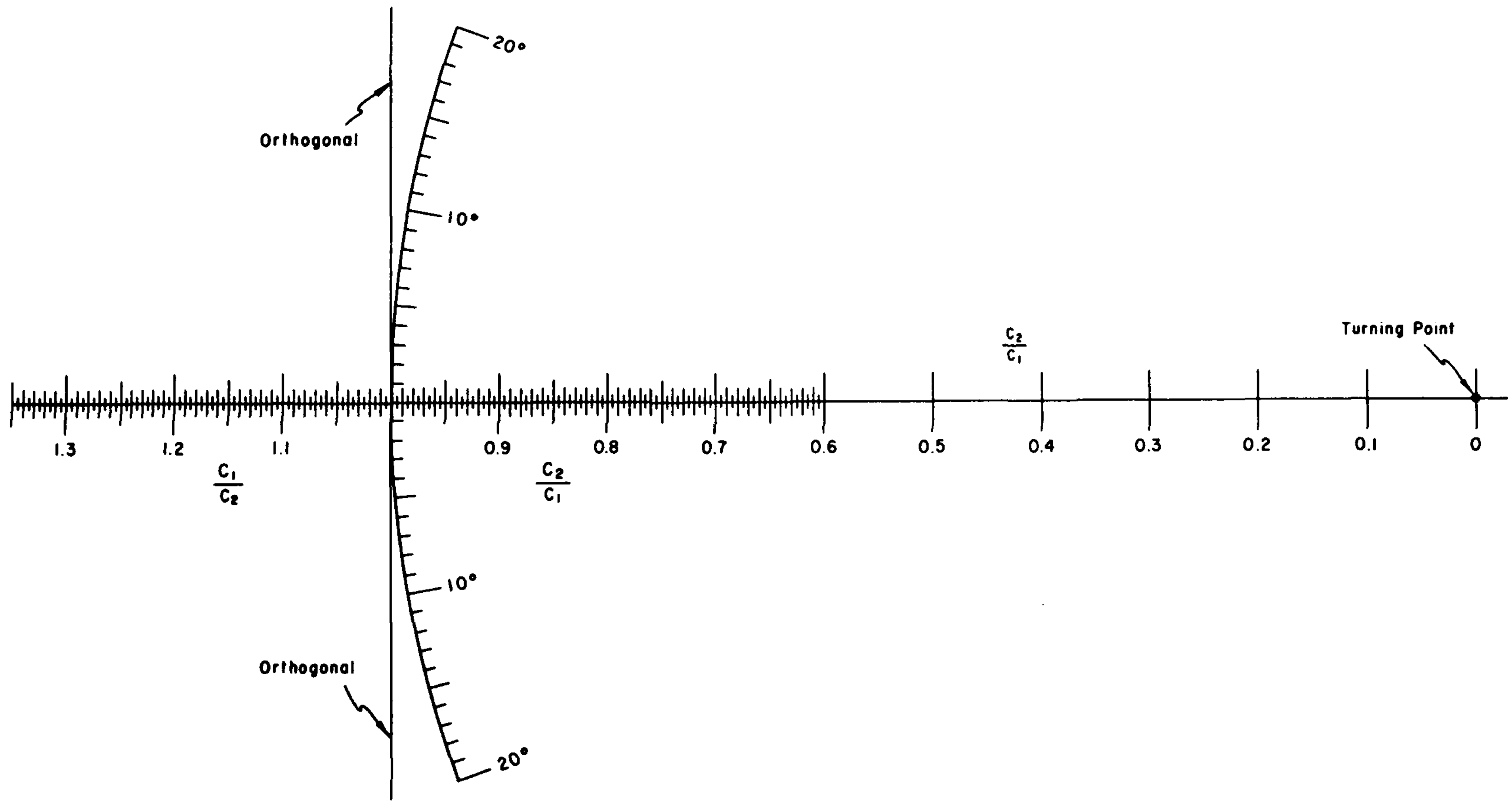


Figure 5-7. Refraction template.

(3) Sketch a contour midway between the first two contours to be crossed, extend the orthogonal to the midcontour, and construct a tangent to the midcontour at this point.

(4) Calculate the value of C_1/C_2 at the midcontour from the equation

$$\frac{C_1}{C_2} = \frac{\tanh(2\pi d_1/L)}{\tanh(2\pi d_2/L)} \qquad (5\text{-}36)$$

where d_1 and d_2 are the depths of the contours on either side of the midcontour, and the value of $\tanh(2\pi d_i/L)$ is obtained from Figure 5-4 or Appendix C as a function of d_i/L_o.

(5) Lay the line on the template labelled orthogonal along the incoming orthogonal with the point 1.0 at the intersection of the orthogonal and midcontour (see Figure 5-8a).

(6) Rotate the template about the turning point until the value of C_1/C_2 obtained in Step 4 intersects the tangent to the midcontour. The orthogonal line on the template now lies in the direction of the turned orthogonal (Figure 5-8b).

(7) The turned orthogonal is now drawn on the chart parallel to the orthogonal line on the template and at a position where the length of the orthogonals, from the contours to the intersection of the incoming and turned orthogonals, are equal. Note that the point of intersection is not necessarily on the midcontour line.

(8) Repeat Steps 3 through 7 for successive contour lines.

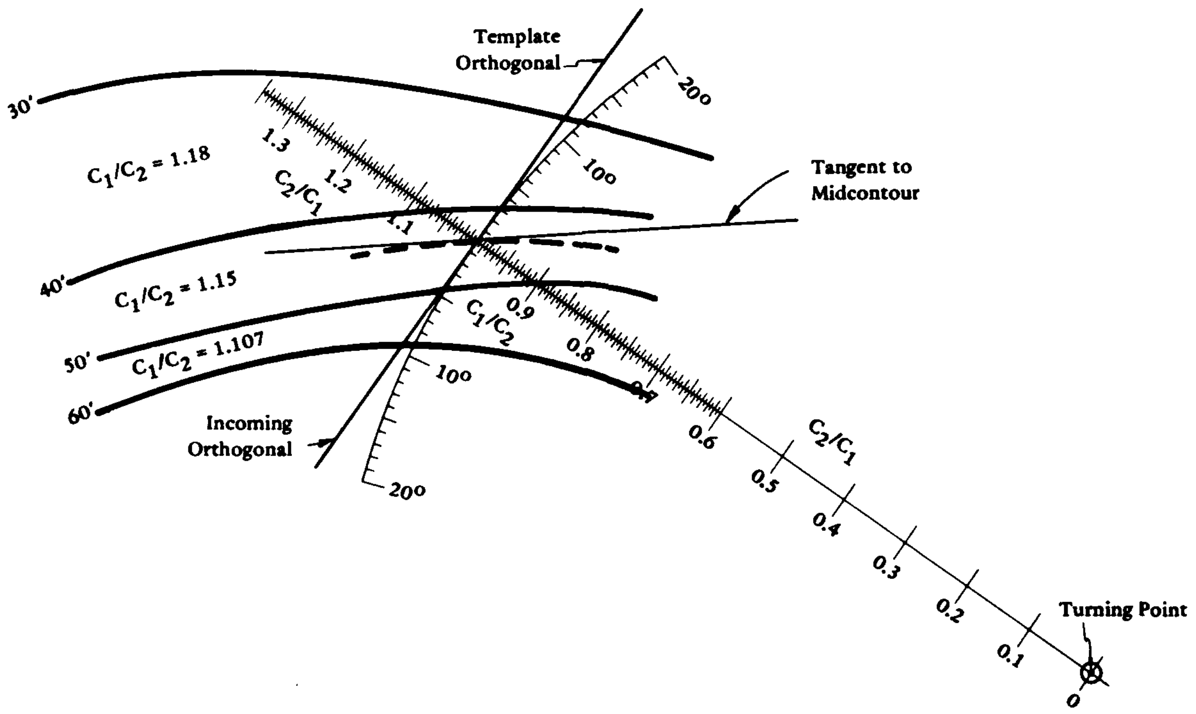


Figure 5-8a. Step 5 of refraction procedure.

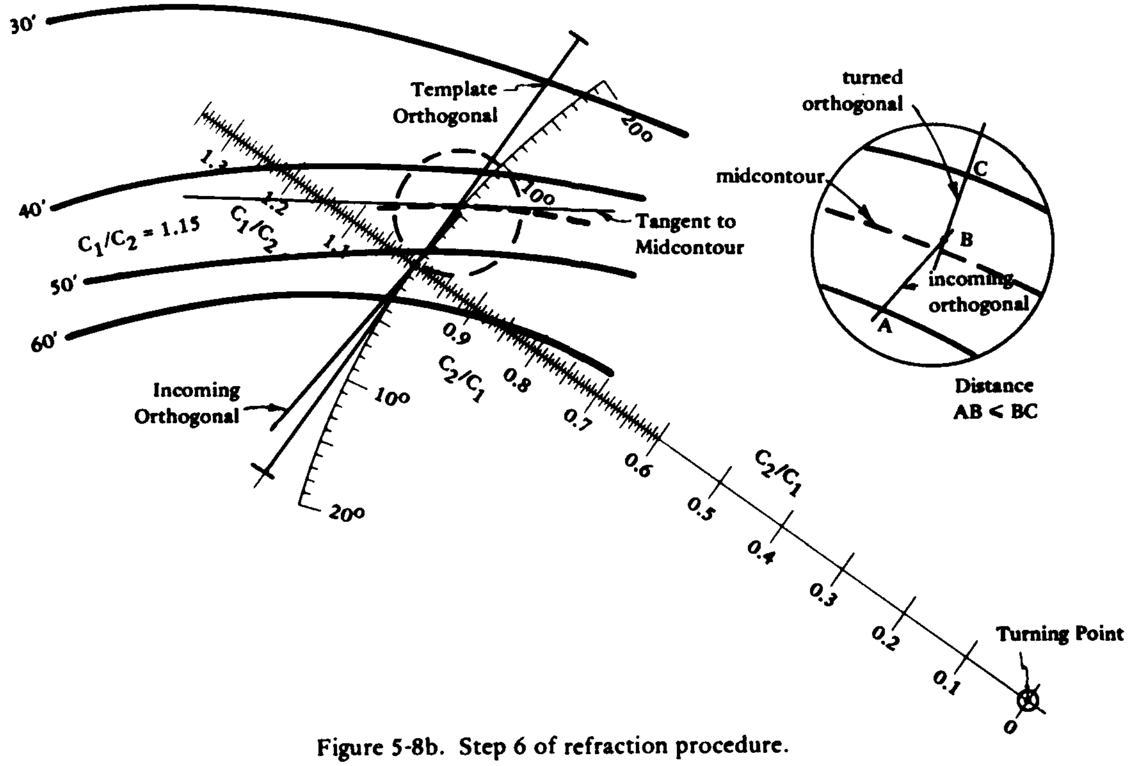


Figure 5-8b. Step 6 of refraction procedure.

When the refraction diagram has been constructed for the entire area of the cable path, the angle α between the velocity and acceleration vectors (u and du/dt) and the cable at any point is obtained by measuring the angle between the orthogonal and the cable path at that point. The refraction coefficient can also be obtained from the chart or Equation 5-32.

5.4.6 Location and Height of Breaking Waves

The theories used for the kinematic analysis of waves are not valid for a wave that has broken; therefore, it is important to define the surfzone region. Experiments have shown that the water depth at which a wave will break (d_b) is a function of the slope of the seafloor (m) and the deep water wave height (H_o) and period (T). The wave height at the point of breaking (H_b) has also been found to be a function of these same three parameters. Figures 5-9 and 5-10 show the results of experiments conducted by Iversen (1953) and Goda (1970). From Figure 5-9, the height of the breaking wave (H_b) can be found knowing the beach slope, deep water wave height, and period. Using this value, the water depth at which the wave will break is obtained from Figure 5-10. Seaward of this breaking wave depth, the wave forces are calculated from the equations presented in Sections 5.4.1 and 5.4.2. From the breaking point shoreward, the height of a wave that would just be about to break is calculated from the constant d_b/H_b obtained from Figure 5-10. The wave forces in the surfzone are then estimated using this imaginary wave height in Equations 5-25 and 5-26 and multiplying the drag coefficient by 2.5 as discussed in Section 5.4.1.

5.4.7 Combined Wave and Current Forces

Hydrodynamic forces produced by current and wave velocities are calculated from the same equations; therefore, from the distributive

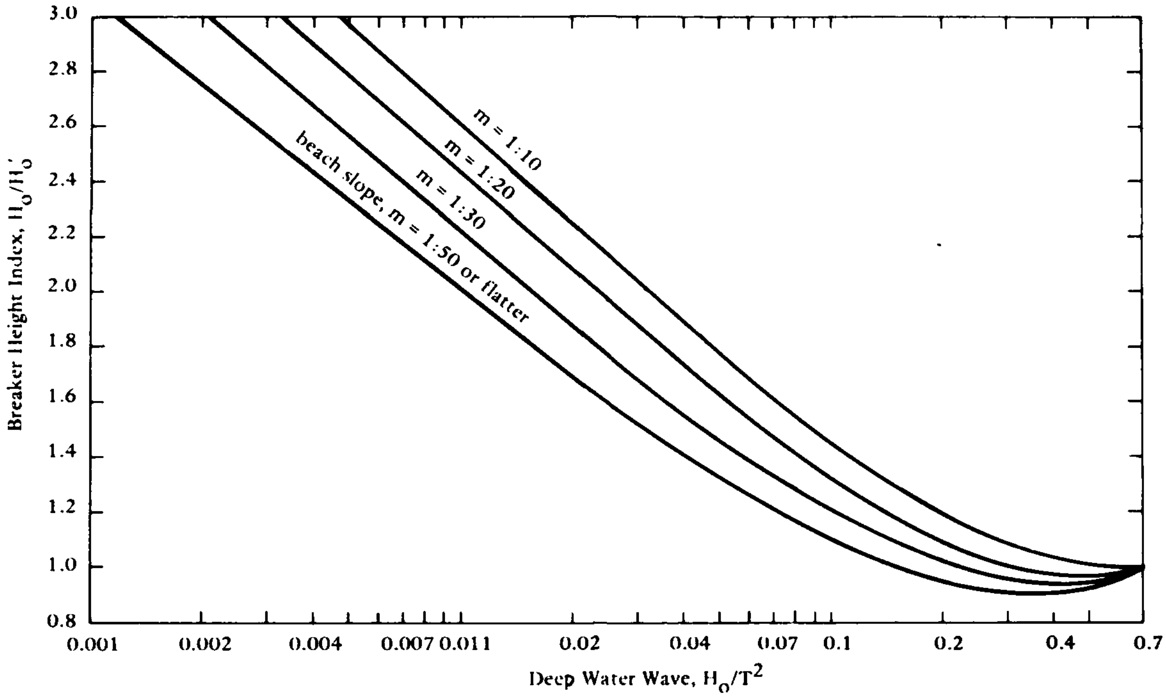


Figure 5-9. Breaker height (after Iversen, 1953).

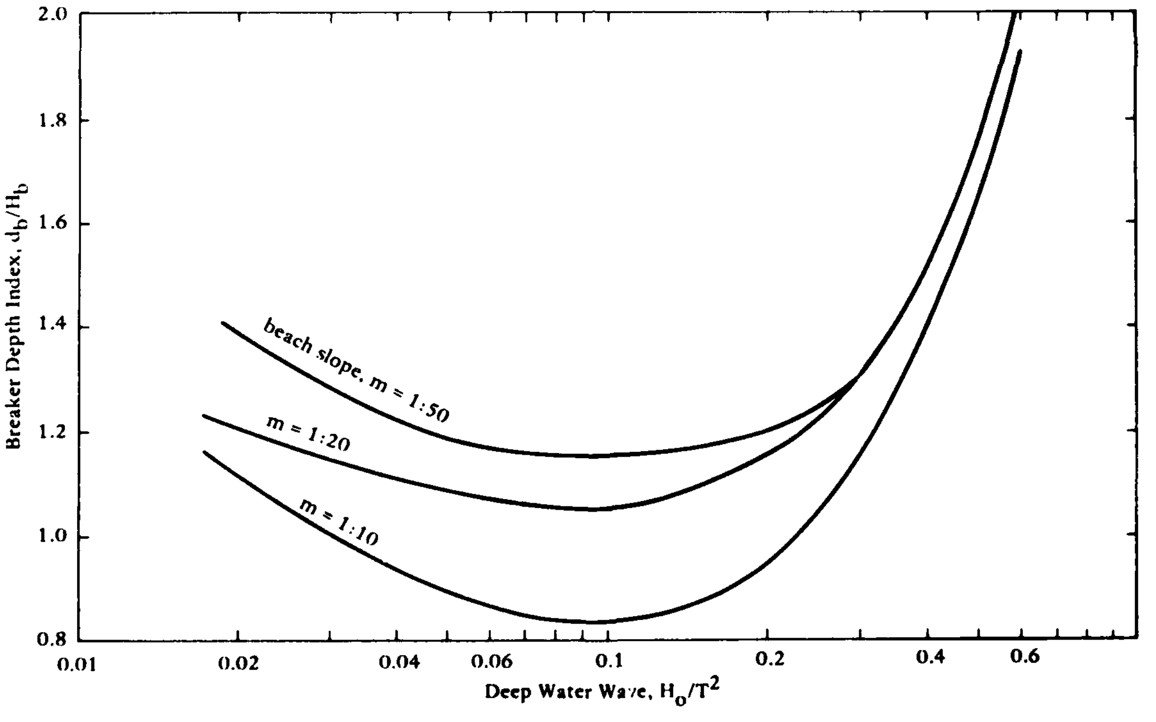


Figure 5-10. Breaking wave depth (after Iversen, 1953).

law, the combined effects of these two forces can be obtained by calculating the single force resulting from the combination of velocity fields. Since velocity is a vector quantity, the resultant velocity will have both magnitude and direction as obtained from the following equations:

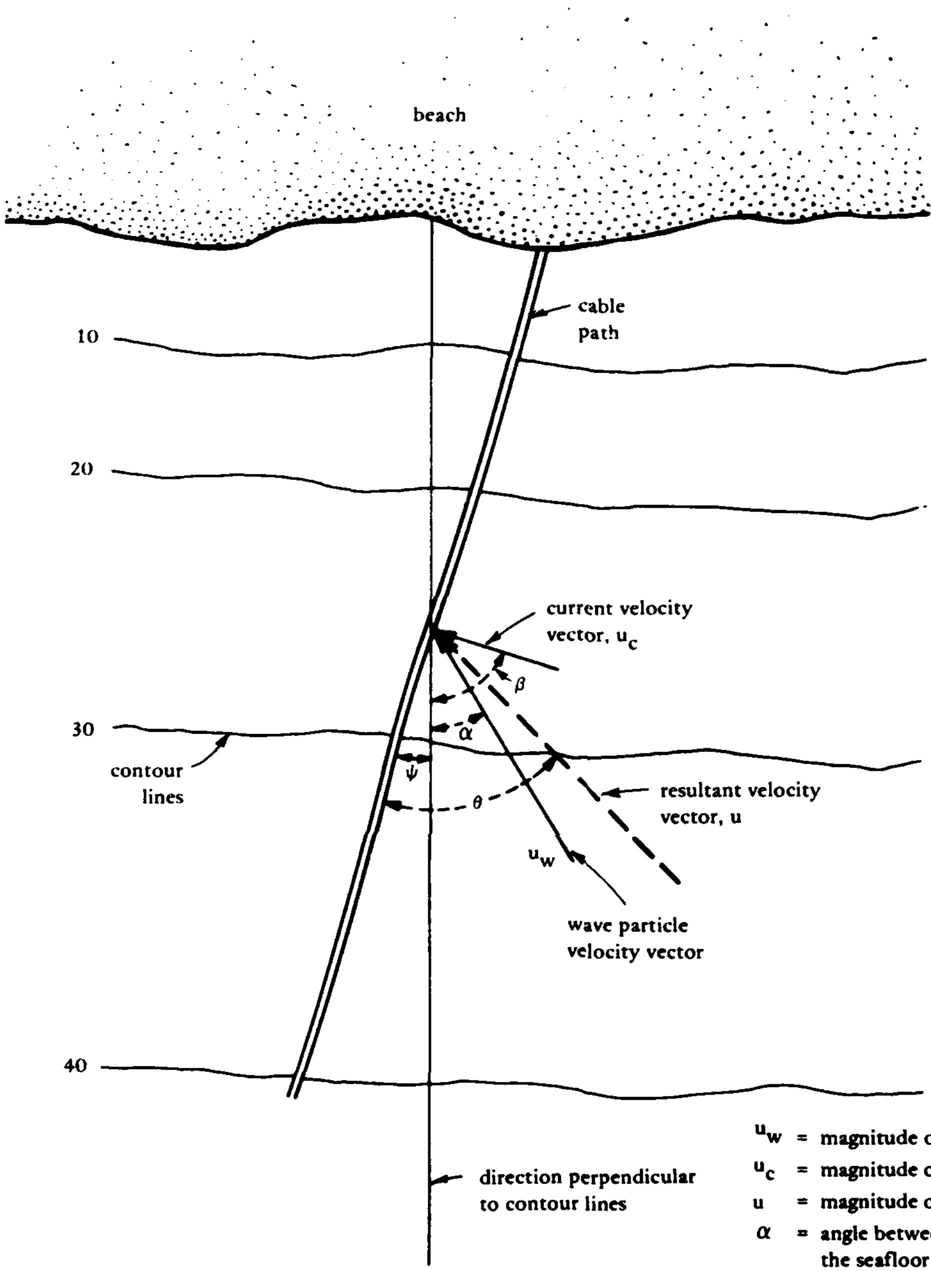
$$u^2 = (u_w \sin\alpha + u_c \sin\beta)^2 + (u_w \cos\alpha + u_c \cos\beta)^2 \qquad (5\text{-}37)$$

$$\theta = \psi + \tan^{-1}\left(\frac{u_w \sin\alpha + u_c \sin\beta}{u_w \cos\alpha + u_c \cos\beta}\right) \qquad (5\text{-}38)$$

Figure 5-11 defines the quantities. These relationships are not absolute, since currents can cause refraction and change the velocity, length, and steepness of waves. For the purpose of this design guide, however, this effect has been neglected. For the reader wishing to pursue this phenomenon further, a detailed discussion is presented by Wiegel (1964).

5.5 FORCE COEFFICIENTS

Selection of the proper values of C_D, C_L, and C_I is perhaps the most controversial part of calculating wave forces on cables. Since these coefficients are empirically determined by substituting experimental results into theoretical equations, their validity depends on the appropriate choice of a wave theory to describe the water particle kinematics and on how well the modeled environment resembles the actual site. These coefficients have also been found to be a function of: (1) type of flow (steady or oscillating), (2) roughness of both the seafloor and cable, (3) angle of incidence of the wave or current with respect to the cable, and (4) clearance between the cable and the seafloor.



u_w = magnitude of wave velocity
u_c = magnitude of current velocity
u = magnitude of combined velocity
α = angle between line perpendicular to the seafloor contours and the wave velocity vector
β = angle between line perpendicular to the seafloor contours and the current velocity vector
ψ = angle between contour perpendicular and the cable path
θ = angle between cable path and resultant velocity vector

Figure 5-11. Definition of terms used in Equations 5-37 and 5-38.

5.5.1 Flow Conditions

Steady Flow. When a cable or mass anchor system is placed in a current where the velocity is essentially constant with respect to time, the drag coefficient has been found to be a function of the Reynolds Number, which is a dimensionless parameter defined as:

$$R_e = \frac{u\,D}{\nu} \tag{5-39}$$

where u = fluid velocity (ft/sec)

D = cable diameter (ft)

ν = kinematic viscosity (approximately equal to $1\mathrm{x}10^{-5}$ ft^2/sec)

The drag coefficient for a smooth cylinder in a free stream varies with Reynolds Number as shown in Figure 5-12. The value of C_D is relatively constant (about 1.2) for R_e less than $1\mathrm{x}10^5$ (subcritical range), decreases from $1\mathrm{x}10^5$ through $4\mathrm{x}10^5$ (critical range), and again is relatively constant (about 0.7) for R_e greater than $4\mathrm{x}10^5$ (supercritical range).

The relationship between the lift coefficient and Reynolds Number does not appear to be as simple. Depending on the research cited, the ratio of C_D/C_L varies between 1/3 and 4 over the range of Reynolds Number.

Oscillating Flow. When a cable encounters an oscillating flow field, such as that produced by wave motion, the Reynolds Number is not always a good criterion for establishing force coefficients (Wiegel, 1964). In 1958, Keulegan and Carpenter established a period parameter defined as

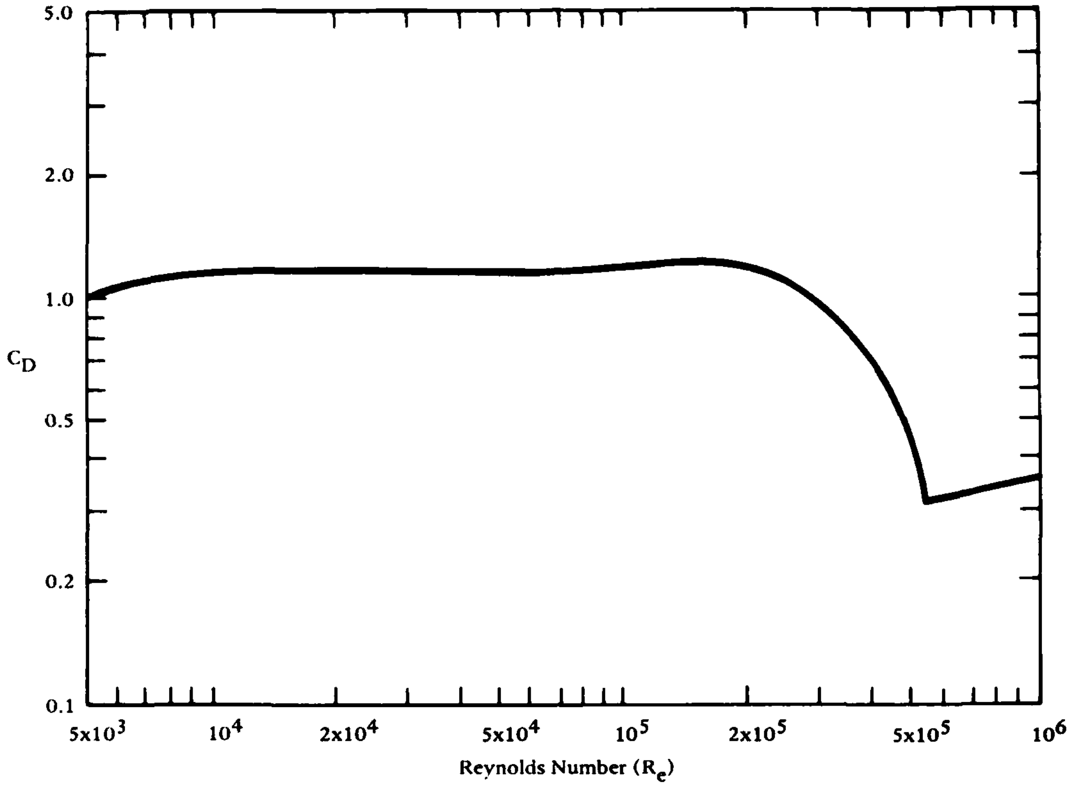


Figure 5-12. Drag coefficients for circular cylinders remote from any boundary; smooth cylinders (after Schlichting, 1960).

$$K = \frac{u_m T}{D} \tag{5-40}$$

where K = Keulegan-Carpenter period parameter

u_m = fluid velocity (maximum)

T = wave period

D = diameter of cable or mass anchor system

This parameter has been generally accepted as a much better indicator of the force coefficients for oscillating flow than the Reynolds Number. Figures 5-13 through 5-15 show the empirically derived values of C_D, C_L, and C_I as a function of K for a smooth cylinder in oscillating flow.

5.5.2 Effect of Surface Roughness

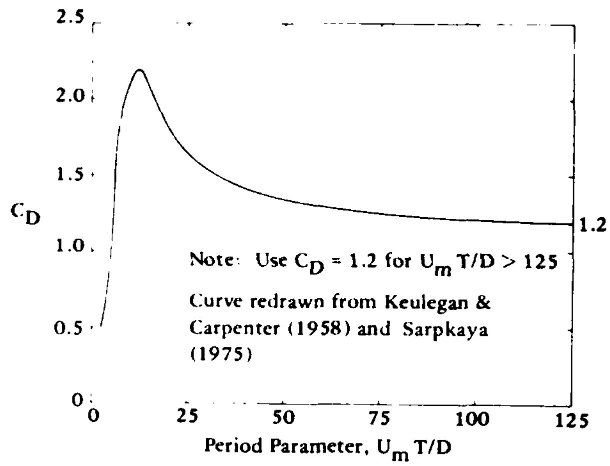


Figure 5-13. C_D versus period parameter for a smooth cylinder (from Davis and Ciani, 1976).

In 1977, Sarpkaya et al. (1977) presented the results of work with rough cylinders in an oscillating flow field. It was found that C_D and C_I depended on Reynolds Number (R_e), the Keulegan-Carpenter parameter (K), and the relative roughness parameter (k/D), where k is the roughness height. C_L, however, was found to be a function of only the period parameter (K). Sarpkaya also established a new parameter that he termed the roughness Reynolds Number given by the following

$$R_e^K = R_e \frac{k}{D} = \frac{u_m k}{\nu} \tag{5-41}$$

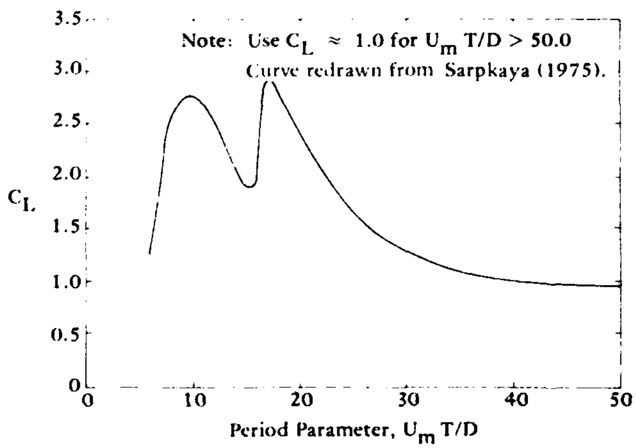


Figure 5-14. C_L versus period parameter for a smooth cylinder (from Davis and Ciani, 1976).

Values of C_D and C_I for various values of k/D are plotted against this parameter in Figure 5-16. From this graph it can be seen that for sufficiently large values of R_e^K, both C_D and C_I become independent of the ratio k/D and are determined by the height of the surface roughness alone rather than the diameter of the cable.

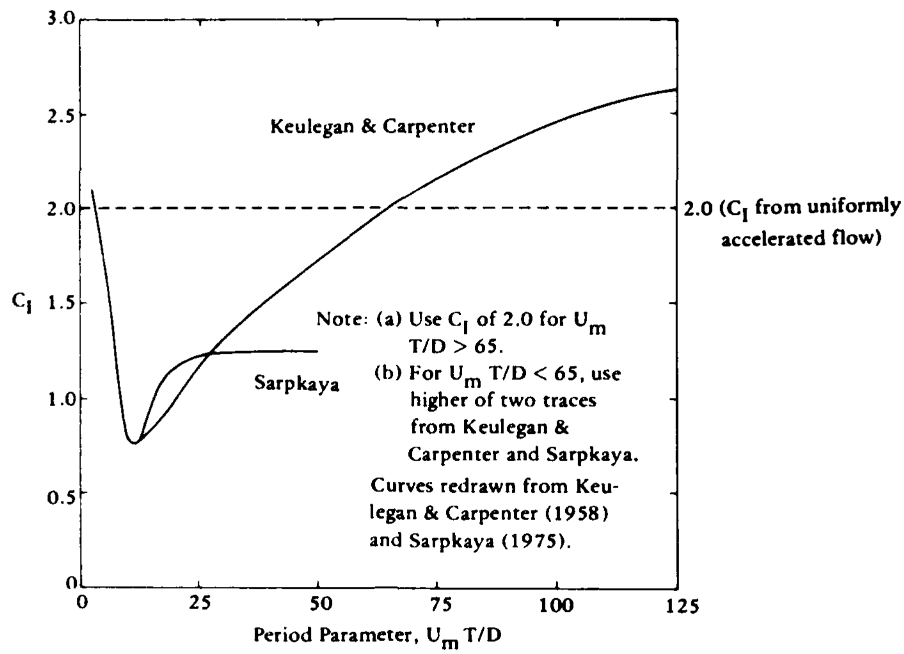
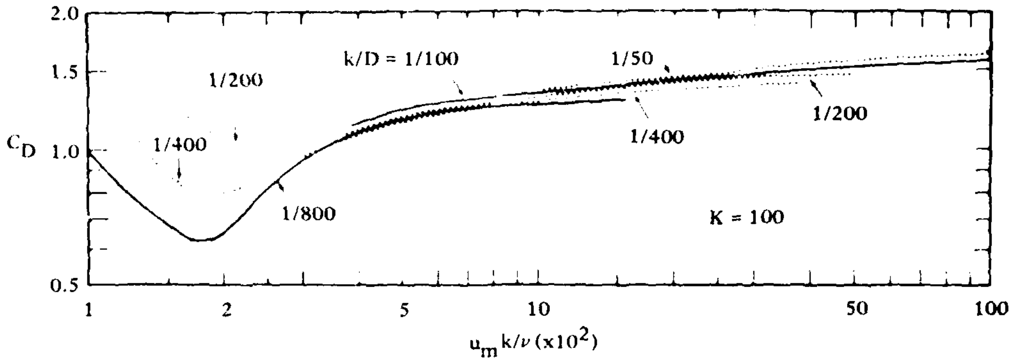


Figure 5-15. C_I versus period parameter for a smooth cylinder (from Davis and Ciani, 1976).

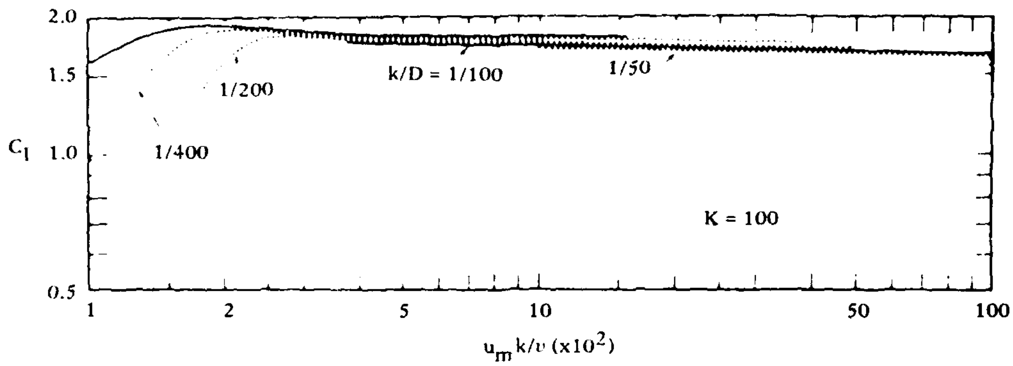
5.5.3 Base Coefficients

In order to account for the effects of the angle of incidence of the velocity field and the clearance of the cable above the seafloor, it has been proposed by Cullison (1975) that base coefficients be established for the flow and roughness conditions anticipated and with the velocity and acceleration fields moving at right angles to the cable path. These base coefficients are then modified by multiplying by correction factors to account for the effects of the influencing parameters.

Table 5-2 presents the base coefficients obtained by evaluating Figures 5-12 through 5-16 for range of values of R_e, K, and k/D found for typical design waves and currents and common cable configurations.



(a) Drag coefficients.



(b) Inertia coefficients.

Figure 5-16. Coefficients of drag and inertia versus roughness Reynold's number for period parameter, K = 100 (from Sarpkaya, 1977).

5.5.4 Effect of Angle of Incidence

Figure 5-17 shows the effect of angle of incidence of the velocity vector on the value of C_D and C_L as presented by Grace (1971). The parameters K_D^θ and K_L^θ are correction factors which are multiplied by the base coefficients C_D' and C_L' to obtain the appropriate force coefficients. This approach was selected over the trigonometric method

described in Section 5.3.1 because it is based on empirical data and, for angles greater than 30 degrees, provides a more conservative design solution.

Table 5-2. Base Coefficients

Surface Condition	Flow Condition	Base Coefficients for –		
		C'_D	C'_L	C'_I
Smooth	Oscillating	1.2	1.0	2.0
	Steady	1.2	0.5	0
Armored cable	Oscillating	1.6	0.25	1.6

5.5.5 Effect of Clearance Above the Seafloor

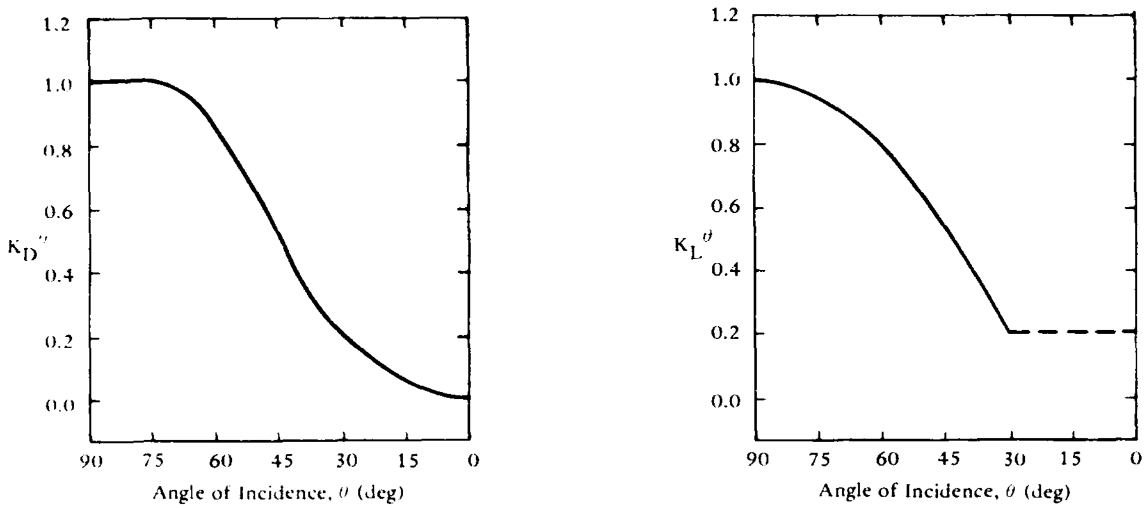
Figure 5-18 shows the effect of clearance between the seafloor and the cable on the values of C_D and C_L as presented by Grace (1971). The values of the correction factors $K_D{}^C$ and $K_L{}^C$ are plotted against the dimensionless parameter e/D rather than the absolute clearance, where e is the clearance between the bottom of the cable and the seafloor and D is the cable diameter.

5.5.6 Combined Coefficients

The effective force coefficients are obtained by combining the base coefficients (Table 5-2) with the correction factors (Figures 5-17 and 5-18) for both angle of incidence and clearance such that

$$C_D = K_D{}^\theta \, K_D{}^C \, C'_D \qquad (5\text{-}42)$$

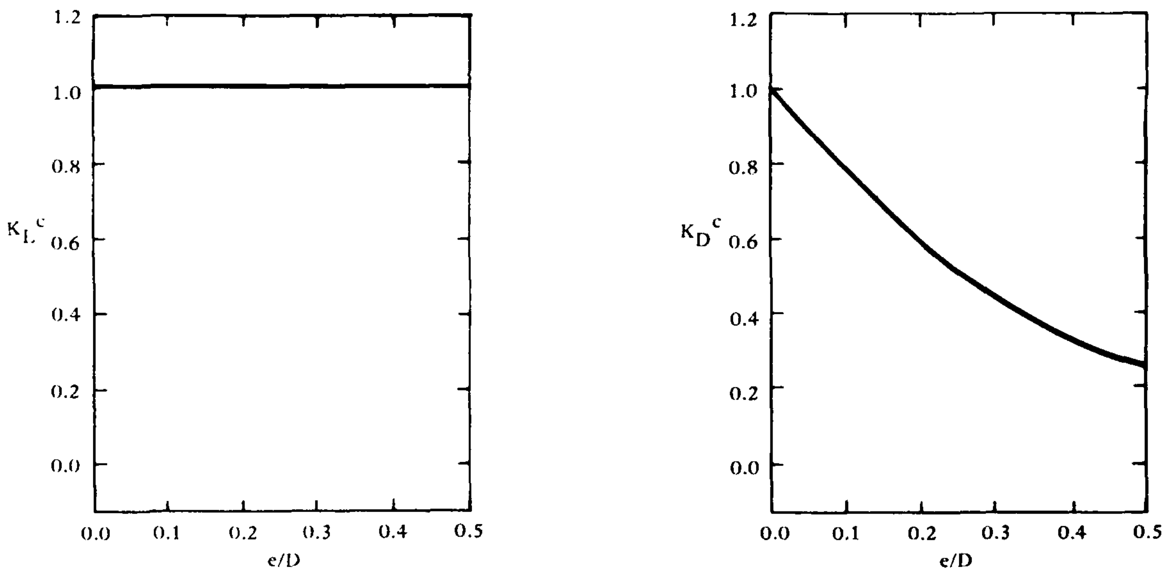
$$C_L = K_L{}^\theta \, K_L{}^C \, C'_L \qquad (5\text{-}43)$$



(a) C_D curve.

(b) C_L curve.

Figure 5-17. Effect of angle of incidence of velocity vector on C_D and C_L (after Cullison, 1975).



(a) C_D curve.

(b) C_L curve.

Figure 5-18. Effect of clearance between seafloor and cable on values of C_D and C_L (after Cullison, 1975).

No data on correction factors were found for the coefficient of inertia and, therefore,

$$C_I = C_I' \tag{5-44}$$

The correction factor K_D^C is found to be unity for all values of e/D. This relationship was developed from work with rigid pipelines and, when applied to flexible cable structures, may not be valid due to the phenomenon of strumming. When a cable is suspended above the seafloor, such that $e/D > 2$, vortices are formed and shed alternately from the top and bottom of the cable, causing an alternating lift force. The frequency of this alternating force is given by the equation:

$$f = S_n \frac{u}{D} \tag{5-45}$$

where
- f = frequency (Hz)
- u = velocity (ft/sec)
- D = diameter (ft)
- S_n = Strouhal Number

A value of $S_n = 0.21$ is recommended by Davis and Ciani (1976) for circular pipes subjected to a steady velocity field; wave motion values have not been reported. When the shedding frequency approaches the natural frequency of the cable system, the drag coefficient can be increased by a factor of up to 2.5 times greater than the nonstrumming coefficient. However, the analysis of the natural frequency of suspended cables is complex, requiring the use of computer programs. Since the cost of obtaining and running such a program is probably in excess of the additional cost to stabilize the end points of the suspension against the increased force, a conservative approach is

to assume that during the life of the system conditions will exist which can cause strumming and a value of $C_D = 2.5\ C_D'$ should be used over the length of the suspension.

5.5.7 Split-Pipe Coefficients

Split pipe has a complex shape (see Figure 4-2) that does not allow the force coefficients obtained for cylindrical bodies to be directly applied. Tests conducted on actual split-pipe sections by Yamamoto (1977) found the force coefficients to be a function of Reynolds Number (R_e), Keulegan-Carpenter parameter (K), and flange angle (ϕ). The flange angle was found to be the most dominant of the three parameters over the range of values investigated.

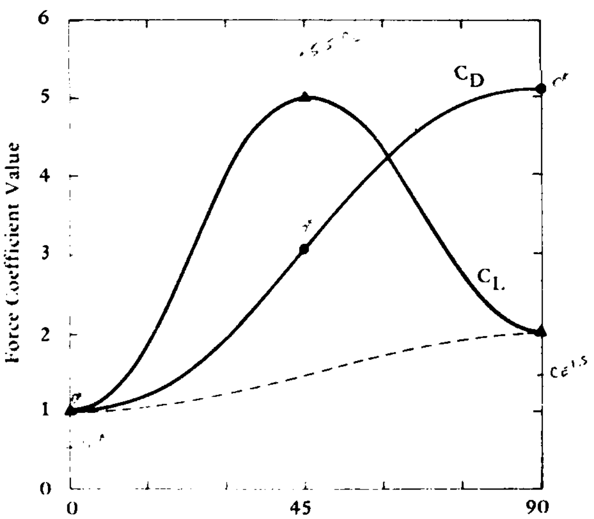


Figure 5-19. Force coefficients versus ϕ for K = 25.

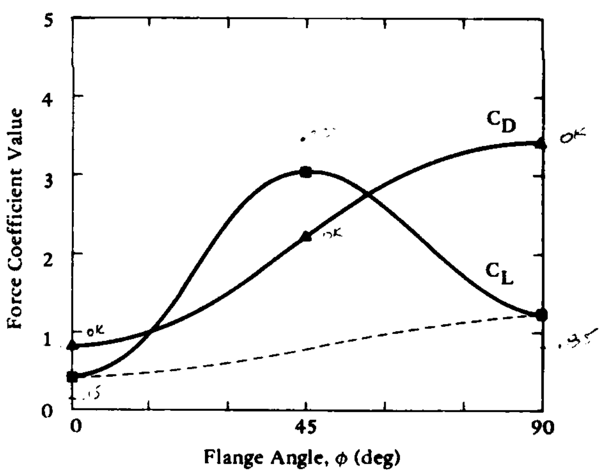


Figure 5-20. Force coefficients versus ϕ for K = 50.

Figures 5-19 and 5-20 were developed from the data presented by Yamamoto (for values of K equal to 25 and 50) along with the assumption that the force coefficients varied sinusoidally with ϕ. Selection of

the base coefficients from these data is difficult since the flange angle can vary from section to section of pipe and will usually change during the life of the installation unless each section is individually restrained.

The worst case design conditions, however, can be established from the following procedure:

(1) Rewrite the stability equation (6-3b) in terms of C_D and C_L; the system will be stable for the condition

$$C_D + \mu C_L < \frac{2 \mu W_s^*}{\rho A u_{max}^2} \tag{5-46}$$

For this equation, the worst case condition occurs for a maximum value of $(C_D + \mu C_L)$.

(2) Express the curves presented in Figures 5-19 and 5-20 in terms of trigonometric functions:

$$C_L = -A \cos(4\phi) - B \cos(2\phi) + C \tag{5-47}$$

$$C_D = -D \cos(2\phi) + E \tag{5-48}$$

where the coefficients depend on the value of K as shown below:

K	A	B	C	D	E
25	2.25	0.5	3.25	2.05	3.05
50	1.425	0.35	1.925	1.3	2.1

(3) Substitute these equations for C_D and C_L into Equation 5-46 and differentiate with respect to ϕ; the critical flange angle can be found from the equation:

$$\phi_{cr} = \sin^{-1}\left(\frac{D + \mu B + 4 \mu A}{8 \mu A}\right)^{1/2} \tag{5-49}$$

(4) Evaluate Equation 5-49 for the coefficients given in Step 2 and various values of μ; the critical flange angle as a function of μ is plotted in Figure 5-21. It should be noted that within the accuracy of the experimental data, ϕ_{cr} is not a function of K.

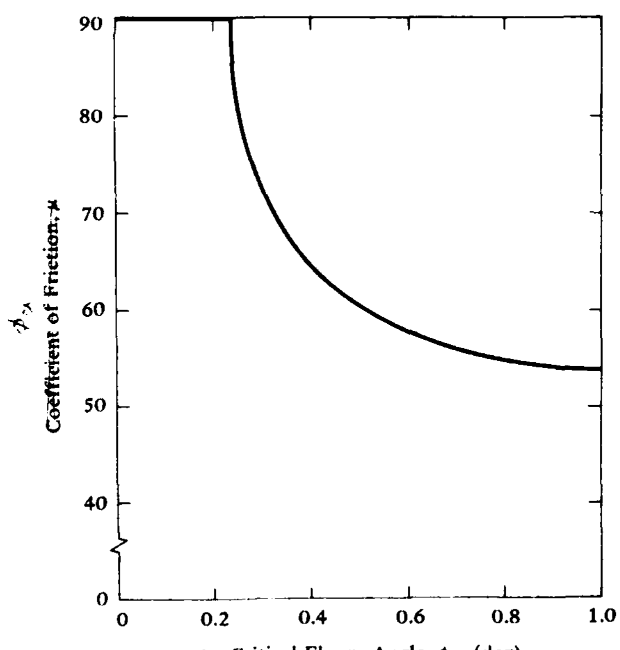


Figure 5-21. Split-pipe critical flange angle versus coefficient of friction.

(5) The worse case design condition is obtained by entering Figure 5-21 for the appropriate coefficient of friction and determining the critical flange angle. The worse case value of $C_D + C_L$ corresponding to this flange angle is then obtained from either Figure 5-19 or 5-20.

Chapter 6

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Chapter 6

REACTION OF OCEAN CABLES TO HYDRODYNAMIC FORCES

6.1 INTRODUCTION

The external forces exerted on ocean cables in the nearshore zone are discussed in Chapter 5 of this design guide. In most realistic design situations, lift and drag forces tend to be the most significant of the hydrodynamic forces causing motion of the cable system. If unrestrained, these forces will cause the cable to roll or slide across the seafloor, resulting in abrasion, kinking, and eventually failure.

When the cable is prevented from rolling, the drag force is resisted by friction between the cable and the seafloor, and sliding will not occur as long as the drag force does not exceed the maximum static friction force. When the drag force does exceed the maximum friction force, the cable will begin to move and is considered unstable. The cable will continue to move under the influence of the drag force until the combination of internal tension forces and external pinning reaction forces again result in an equilibrium condition. This new equilibrium position is only stable for a short time, however, since wave forces are oscillatory in nature and change both in magnitude and direction as each wave passes over the cable. The displacement of a cable between seafloor fasteners is a function of the stiffness of the cable and pins, the spacing between the pins, friction, and initial tension of the cable. Although it is theoretically impossible to completely eliminate all movement of an unstable cable, proper selection and spacing of seafloor

fasteners and optimum pretensioning of the cable can reduce the magnitude of the deflection to the point where it will not result in critical damage or failure.

The friction force that resists the drag force is a function of the coefficient of friction between the cable (or mass anchor) and the seafloor and the vertical reaction force between the seafloor and the cable system. Since the lift force acts in the opposite direction to the weight of the cable, it reduces the magnitude of the friction force and, therefore, causes the cable to become unstable for lower drag forces.

The value selected for the coefficient friction will depend on the material of the seafloor and cable system component in contact with the seafloor, the amount of marine growth, roughness of the seafloor (which can cause interlocking of the cable and seafloor), and the condition of the cable. Precise values for the coefficient of friction have not been tabulated and should be the subject of further research. In the absence of absolute values, both Hudspeth (1972) and Cullison (1975) citing the Army CERC (1973) recommended using $\mu = 0.3$ for iron on hard stone.

When wave orthogonals approach a cable at an angle other than 90 degrees, both the magnitude and direction of the hydrodynamic force will vary along the length of the cable. Analysis of the stability of cables under such a force function is difficult, at best. Therefore, for the purposes of this design guide, the cable will be assumed to consist of finite lengths that can be considered to experience uniform loading. The assumption will also be made that the seafloor is smooth (i.e., no interlocking of the cable and seafloor) and that the cable system is sufficiently restrained so that it does not roll or twist.

When a portion of the cable is suspended above the seafloor, that length of the cable is inherently unstable since no friction force can be developed to restrain it. Under the influence of wave loads, a

suspended cable will swing back and forth at a frequency equa. .o the wave frequency. If the low point of the cable suspension should come in contact with the seafloor, severe abrasion and ultimately failure will generally result. Since cable installations are planned to avoid large suspensions, this factor will not be considered in the initial design of the stabilization system. However, once installed, the presence of any suspensions must be investigated and their effect on the integrity of the cable system determined.

6.2 STABILITY OF CABLES AND MASS ANCHOR SYSTEMS

When a cable is stabilized by a mass anchor system (Section 4.2), the hydrodynamic forces are resisted only by friction and, therefore, the submerged weight, projected area, and hydrodynamic force coefficients are the critical system parameters affecting stability. Figure 6-1 shows a free body diagram of a cable resting on the seafloor and the forces which act on it. The stability equations for a unit length free body are:

$$F_N^* = \begin{cases} W_s^* - F_L^* & \text{for } F_L^* < W_s^* \\ 0 & \text{for } F_L^* > W_s^* \end{cases} \tag{6-1}$$

where W_s^* = $W^* - B^*$ = weight (in water) per unit length

F_N^* = seafloor reaction force per unit length

F_L^* = lift force per unit length

W^* = weight (in air) per unit length

B^* = buoyant force per unit length

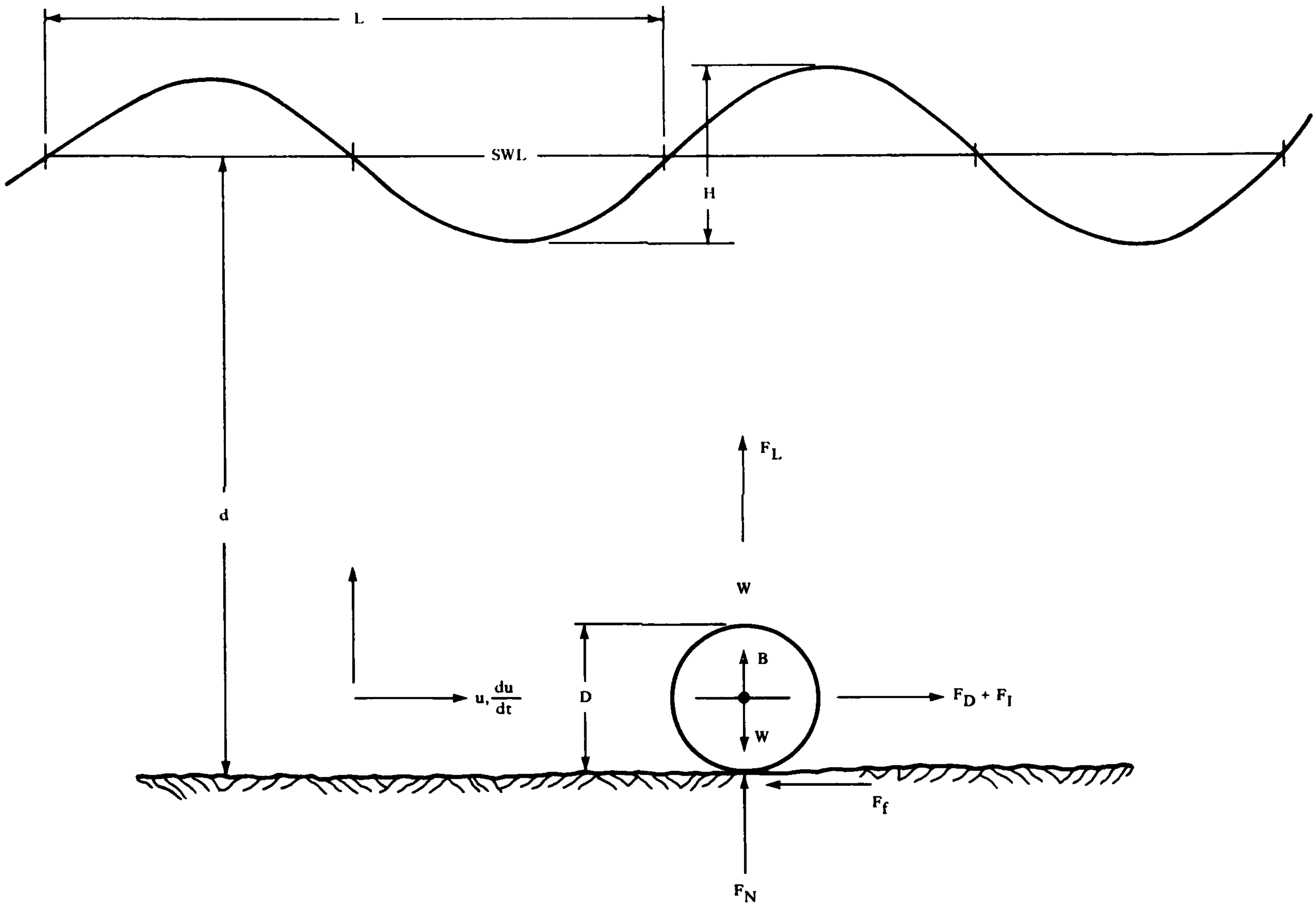

Figure 6-1. Free body force diagram for bottom-resting cables.

and

$$F_H^* = F_D^* + F_I^* - \mu F_N^* \tag{6-2}$$

where F_H^* = net horizontal force per unit length

F_D^* = drag force per unit length

F_I^* = inertia force per unit length

μ = coefficient of friction

F_N^* = seafloor reaction force per unit length

Substituting Equations 5-15 through 5-17 into Equation 6-2, the horizontal stability equation for a cable/mass anchor system may be expressed by:

$$F_H^* = \frac{1}{2} C_D \rho_w D u^2 + \frac{\pi}{4} C_I \rho_w D^2 \frac{du}{dt} - \mu \left(W_s^* - \frac{1}{2} C_L \rho_w D u^2 \right) \tag{6-3a}$$

For most design situations where the inertia component is insignificantly small, Equation 6-3a can be reduced to:

$$F_H^* = \frac{1}{2} (C_D + \mu C_L) \rho_w D u^2 - \mu W_s^* \tag{6-3b}$$

When the stability criteria is applied to Equation 6-3a or 6-3b, the cable system is considered stable for the condition:

$$F_H^* \leq 0$$

and unstable for the condition:

$$F_H^* > 0$$

6.3 IMMOBILIZATION OF OCEAN CABLES WITH SEAFLOOR FASTENERS

6.3.1 Reaction of Immobilized Cables to Hydrodynamic Forces

When the condition $F_H^* > 0$ exists for a cable/mass anchor system, it is considered unstable and will move under the influence of design wave and current forces. This movement (deflection) can be controlled by immobilizing the cable with seafloor fasteners at various points along its length (Section 4.3). Although, theoretically, it is impossible to completely eliminate all motion of an unstable cable by pinning it to the seafloor, the proper design of a seafloor fastener system can reduce the magnitude of this motion to the point where it will not cause failure during the life of the system.

The analysis of immobilized cables was developed using the following assumptions:

(1) The cable is sufficiently flexible so that it can only transmit axial loads.

(2) The cable is restrained so that it cannot twist about its axis.

(3) All axial forces are transmitted through the armor wires (i.e., the core does not contribute to the strength of the cable).

(4) The cables does not roll along the seafloor.

(5) The ratio of the deflection of the midpoint (δ) of the cable to the span (L_c) (length between immobilization points) is small (i.e., $\delta/L_c << 1$).

(6) Internal stresses produced in the cable do not exceed the elastic limit.

For a cable conforming to the above assumptions, which has been immobilized by seafloor fasteners spaced a distance (L_c) along the length of the cable, the net horizontal hydrodynamic force per unit length of cable (F_H^*) will produce a deflection (δ) at the midspan of the stabilization points as shown in Figure 6-2.

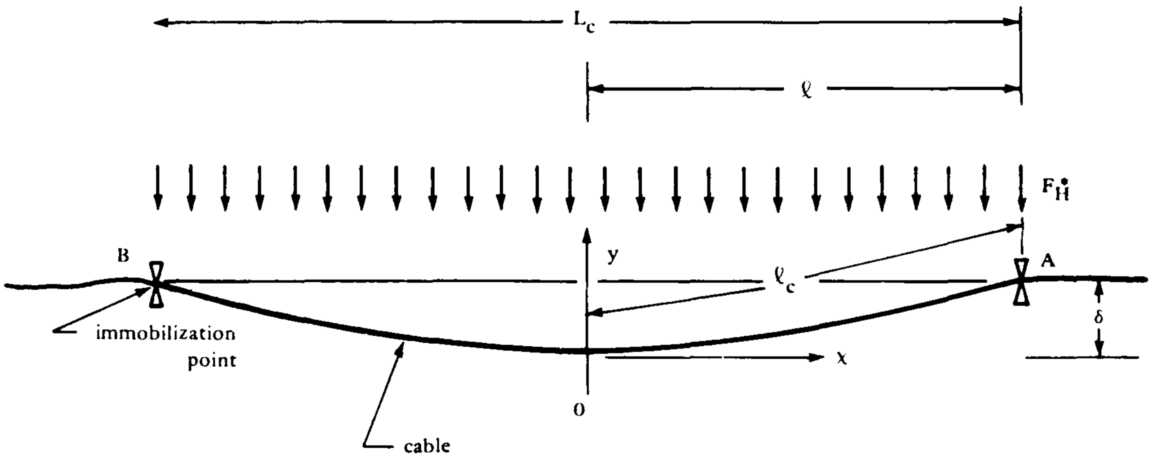

Figure 6-2. Diagram of cable immobilized by seafloor fasteners.

For a uniformly distributed horizontal force, the cable assumes a parabolic shape (Beer and Johnson, 1962), and the maximum deflection is given by the equation:

$$\delta = \frac{F_H^* \ell^2}{2(T_I + T_\Delta)} \tag{6-4}$$

where δ = deflection of the cable at midspan

F_H^* = net horizontal hydrodynamic force per unit length of cable (from Equation 6-3a or 6-3b)

ℓ = half span length = $L_c/2$

L_c = span length

T_I = initial tension in cable

T_Δ = tension in cable due to strain

When a cable is laid on the seafloor without any slack, an equilibrium configuration can be achieved only if:

(1) The length of the cable between the immobilization points increases due to internal stresses, and/or

(2) The immobilization point deflects as a result of the applied reaction forces.

If the deflection to half span ratio (δ/ℓ) is small, then the internal tension can be considered to be uniform. Under the influence of the internal strain induced tension (T_Δ), the portion of the cable from 0 to A will elongate an amount Δ_1, given by:

$$\Delta_1 = \frac{4\, T_\Delta\, \ell}{\pi\, E_c\, D^2} \tag{6-5}$$

where Δ_1 = elongation of the cable

T_Δ = tension in the cable due to strain

ℓ = half span length

E_c = effective modulus of elasticity of the cable

D = diameter of the cable

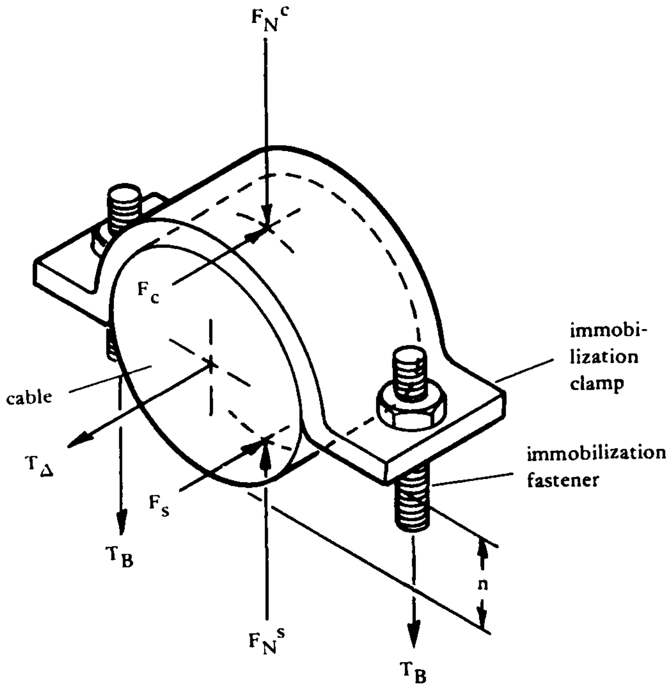

Figure 6-3. Typical immobilization system for cable.

6.3.2 Reaction of Immobilization Systems to Cable Loads

A typical cable immobilization system, shown in Figure 6-3, consists of a clamp, the cable, and the seafloor fasteners. Pretension of the seafloor fastener produces a normal force between the clamp and the cable (F_N^c) and between the cable and the seafloor (F_N^s). If the weight of the cable under the clamp is small compared to the compressive force, then:

$$F_N^c = F_N^s = N\,T_B \tag{6-6}$$

where N = number of fasteners per clamp

T_B = pretension load on fastener

F_N^c = normal force between clamp and cable

F_N^s = normal force between cable and seafloor

Section 4.3.3 provides design data on maximum pretension loads that can be applied to various rock bolts in different seafloor materials. If the cable and the seafloor are both much stiffer in shear than the fastener is in bending, then the cable tension force (T_Δ) is resisted only by friction between the seafloor and cable for

$$0 \geqq T_\Delta \geqq \mu_s N T_B \tag{6-7}$$

For this condition, no motion of the cable occurs under the clamp, and the immobilization point is stable. As the tension increases above the limits of Equation 6-7, the immobilization system will begin to deflect due to shear and bending of the seafloor fastener, which allows the cable to move a small distance (Δ_2) along the seafloor. The general equation for this deflection is given by:

$$\Delta_2 = \left[T_\Delta - \mu_s N T_B\right] \left[\frac{h^3}{3 E_B I_B} + \frac{h}{G_B A_B}\right] \tag{6-8}$$

where Δ_2 = deflection of cable and immobilization system due to bending of the seafloor fastener

h = distance between the seafloor and the bottom of the clamp

E_B = modulus of elasticity of the seafloor fastener

I_B = moment of inertia of the cross section of the fastener

G_B = shear modulus of the fastener

A_B = cross-sectional area of the fastener

μ_s = coefficient of friction between cable and seafloor

For a fastener with a circular cross section, Equation 6-8 becomes

$$\Delta_2 = \frac{8\ T\ h}{\pi\ E_B\ d_B^{\ 2}} \left[\frac{8}{3} \left(\frac{h}{d_B} \right)^2 + (1 + \mu_p) \right] \tag{6-9}$$

where d_B = diameter of immobilization bolt or fastener
μ_p = Poisson's ratio
$T = (T_\Delta - \mu_s\ T_B)$

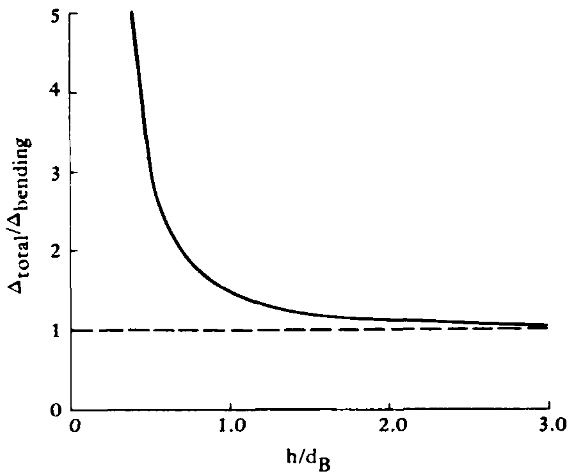

Figure 6-4. Ratio of total fastener deflection to bending deflection versus h/d_B.

Figure 6-4 shows the ratio of total deflection to that produced by bending alone. From this graph it can be seen that the portion of the deflection due to shear becomes insignificant (less than 10% of the total) for $h/d_B > 2$; for design purposes, Equation 6-8 can be reduced to:

$$\Delta_2 = \frac{(T_\Delta - \mu_s\ N\ T_B)\ h^3}{3\ E_B\ I_B} \tag{6-10}$$

The magnitude of this deflection increases linearly as T_Δ is increased until either the cable begins to slip through the clamp at a load:

$$T_\Delta = (\mu_s + \mu_c)\ N\ T_B \tag{6-11}$$

or plastic deformation of the fastener begins to occur at a load:

$$T_\Delta = \mu_s N T_B + \frac{N \pi d_B^3}{16 M h} \sigma_y^B \tag{6-12}$$

where μ_c = coefficient of friction between clamp and cable

$$M = 1 + \left[1 + \left(0.25 \frac{d_B}{h} \right)^2 \right]^{1/2}$$

σ_y^B = yield stress of immobilization bolt

In the limit, as h/d_B approaches ∞, M = 2 and Equation 6-12 becomes

$$T_\Delta = \frac{N \pi d_B^3}{32 h} \sigma_y^B + \mu_s N T_B \tag{6-13}$$

which is the equation for the maximum allowable transverse load if only bending stresses are considered. When h/d_B approaches 0, M converges to $d_B/4h$ and Equation 6-12 reduces to

$$T_\Delta = \frac{N \pi d_B^2}{4} \sigma_y^B + \mu_s N T_B \tag{6-14}$$

which is the maximum shear load that can be applied before yielding occurs. Figure 6-5 shows that for $h/d_B > 1$, bending stresses predominate and Equation 6-13 gives an adequate approximation of the maximum load for design purposes.

6.3.3 Effect of Clamp Design on Deflection and Maximum Load Condition

Four basic clamp configurations, Figures 6-6 through 6-9, have been used in the past for immobilization of ocean cables. The effect of each configuration on the deflection and maximum allowable load of the immobilization system is discussed below and summarized in Table 6-1.

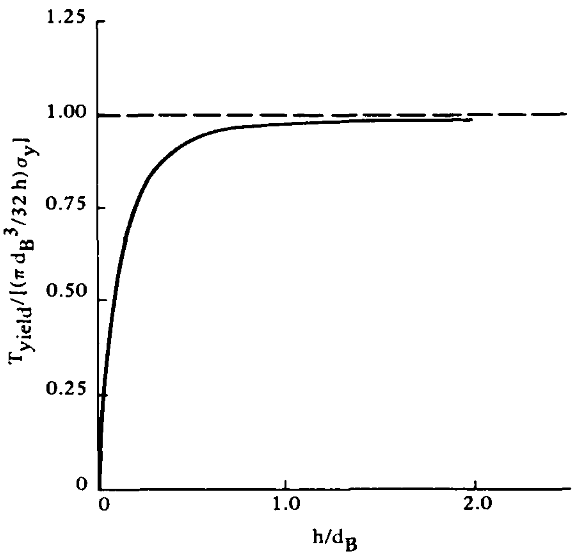

Figure 6-5. Ratio of yield force to maximum bending force versus h/d_B.

(1) The clamp shown in Figure 6-6 is usually constructed from a strip of metal plate bent to conform to the shape of the cable; it is provided with integral tabs or flanges through which the seafloor fasteners are installed. If the width-to-thickness ratio (b/t_c) is large (greater than 10), the clamp will essentially transmit only transverse loads to the fastener at a height h above the seafloor. Since h is variable with the design of the specific clamp, the allowable tension is increased and deflection decreased as the flange stand-off distance approaches zero. The limit h = 0 can never be realized, however,

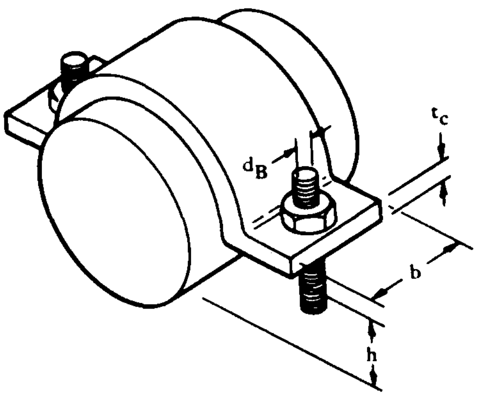

Figure 6-6. Strap clamp.

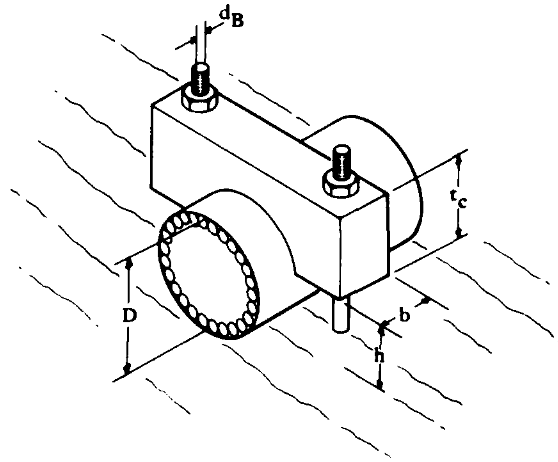

Figure 6-7. Block clamp.

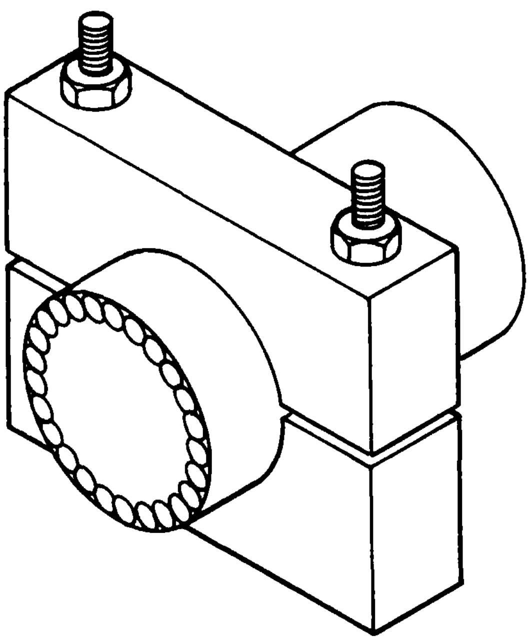

Figure 6-8. Double block clamp.

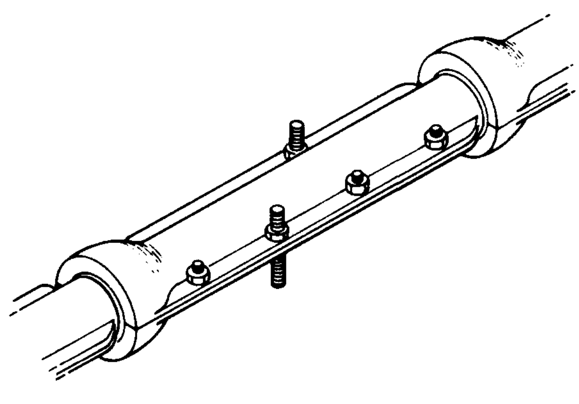

Figure 6-9. Immobilized split-pipe.

without loosing the effect of the pretension load on the cable. The minimum design value for h will be controlled by the roughness of the seafloor along the cable path. In most realistic situations this will result in $h/d_B > 2$, and bending will be the predominant factor for both deflection and tension.

Table 6-1. Effect of Clamp Configuration on Deflection and Maximum Allowable Tension of Cable Immobilization Systems

Configuration	h	Typical h/d_B	Δ_2	$T_{\Delta max}$
1 - Strap Clamp	variable	>2	$\frac{64(T_\Delta - \mu_s N T_B) h^3}{3 E_B \pi d_B^4}$	$\frac{N \pi d_B^3}{32 h} \sigma_y^B + \mu_s N T_B$
2 - Block Clamp	D	>2	$\frac{64(T_\Delta - \mu_s N T_B) D^3}{3 E_B \pi d_B^4}$	$\frac{N \pi d_B^3}{32 D} \sigma_y^B + \mu_s N T_B$
3 - Double Block Clamp	0	0	0	$\frac{N \pi d_B^2}{4} \sigma_y^B + \mu_s N T_B$
4 - Split-Pipe	3.5 in.	5.6	$(6.4 \times 10^{-5}$ in./lb$)(T_\Delta - \mu_B N T_B)$	$N(616$ lb $+ \mu_s T_B)$

(2) Clamps similar to that shown in Figure 6-7 are constructed from blocks of nonmetallic materials to minimize corrosion and are machined to conform to the diameter of the cable. Since the b/t_c ratio is usually less than 1, the clamp transmits both a moment and transverse force to the fastener. The net effect is that of a transverse force applied to the fastener at a distance h = D above the seafloor. For this case, bending stresses and deflections predominate for the design analysis.

(3) The double block clamp shown in Figure 6-8 has the advantage of eliminating bending stresses from the seafloor fastener which results in a nondeflecting immobilization system for loads up to the shear yield stress of the fastener; that is, $\delta = 0$ for

$$T_{\Delta} < \frac{N \pi d_B^2}{4} \sigma_y^B + \mu N T_B$$

This type of clamp is more expensive to machine and more difficult to install; therefore, it has not been used extensively in the past. If additional nuts and bolts are used to hold the clamp together, this type of immobilization system maintains its integrity even if the pretension of the seafloor fastener is lost.

(4) Split-pipe has often served as the clamping mechanism for a cable immobilization system, as shown in Figure 6-9. This type of installation can be considered to be a special case of configuration 1 (Figure 6-6) with h and d_B fixed by the geometry of the pipe.

Since the double block clamp has found limited application in the past and bending stresses and deflections predominate for the other configurations, the remainder of the discussion of immobilization system design theory will be based on the assumption that the immobilization point can be modeled as a cantilevered structure and that Equations 6-10 and 6-13 provide a reasonable approximation of the deflection and maximum allowable force.

6.3.4 Equilibrium of Internal and External Forces

The length of a cable that has a parabolic shape as described by Equation 6-4, can be approximated by the binomial expansion of the integral

$$\ell_c = \int_0^{\ell} \sqrt{1 + \left(\frac{dy}{dx}\right)^2}\, dx$$

to give:

$$\ell_c \approx \ell\left[1 + \frac{2}{3}\left(\frac{\delta}{\ell}\right)^2\right] \tag{6-15}$$

Since ℓ_c is also equal to the original cable length (ℓ) plus the elongation of the cable (Δ_1) plus the deflection of the immobilization system (Δ_2), the total deflection of the system can be approximated by:

$$\delta = \left[\frac{6\ T_\Delta\ \ell^2}{\pi\ E_c\ D^2} + \frac{(T_\Delta - \mu\ N\ T_B)\ h^3\ \ell}{2\ N\ E_B\ I_B}\right]^{1/2} \tag{6-16}$$

For the system to be in equilibrium, the deflection caused by the external hydrodynamic forces (Equation 6-4) must be the same as the deflection caused by the internal forces (Equation 6-16).

6.4 DEVELOPMENT OF IMMOBILIZATION SYSTEM DESIGN THEORY

6.4.1 General Design Equations

Inspection of Equations 6-4 and 6-16 reveals that four variables are involved in the design of a cable immobilization system (T_I, T_Δ, δ, and ℓ), while only two equations are available for simultaneous solution. By solving these two equations to eliminate one of the variables, the resulting equation can be solved if any two of the three remaining variables can be specified. By solving Equation 6-4 for the variables δ, T_Δ, and ℓ, and substituting into Equation 6-16, the following three design equations result:

$$\ell^3 = \psi_\delta (T_I + T_\Delta)^2 \, T_\Delta \qquad (6\text{-}17)$$

$$\delta^2 = \psi_{T_\Delta} \, \ell^2 \qquad (6\text{-}18)$$

$$\delta^2 = \psi_\ell (T_I + T_\Delta) \, T_\Delta \qquad (6\text{-}19)$$

where the ψ's will be defined as the transfer functions for the two remaining variables shown in the equation, and the subscript indicates the variable eliminated by the substitution. The transfer functions are given by the following equations:

$$\psi_\delta = \left[\frac{24 \, \ell}{\pi \, F_H^{*2} \, E_c \, D^2} + \frac{2 \, h^3 \left(1 - \dfrac{\mu_s \, N \, T_B}{T_\Delta}\right)}{N \, F_H^{*2} \, E_B \, I_B} \right] \qquad (6\text{-}20)$$

$$\psi_{T_\Delta} = \left[\frac{6\left(\frac{F_H^* \ell^2}{2\,\delta} - T_I\right)}{\pi\, E_c\, D^2} + \frac{\left(\frac{F_H^* \ell^2}{2\,\delta} - T_I - \mu\, N\, T_B\right) h^3}{2\, N\, E_B\, I_B\, \ell} \right] \tag{6-21}$$

$$\psi_\ell = \left\{ \frac{12\,\delta}{\pi\, E_c\, D^2\, F_H^*} + \left[\frac{2\,\delta}{F_H^*(T_I + T_\Delta)} \right]^{1/2} \left[\frac{h^3 \left(1 - \frac{\mu\, N\, T_B}{T_\Delta}\right)}{2\, N\, E_B\, I_B} \right] \right. \tag{6-22}$$

6.4.2 Design Criteria

Two basic criteria can be used for the design of immobilization systems. The first is based on design of the system so that loads induced in the cable and immobilization system by hydrodynamic forces do not exceed a specified maximum level. The second design procedure is based on the criterion of maximum allowable deflection of the cable at the midspan of the immobilization points. The maximum allowable spacing between immobilization points, calculated for each design criterion, assures that (1) mechanical failure of the system will not occur for a design based on maximum allowable load, and (2) abrasion failure of the cable will not occur for a design based on maximum allowable deflection. In order to assure the survivability of the cable throughout the required design life of the installation, both of these criteria must be satisfied simultaneously. To accomplish this, the immobilization system must be designed for each criterion separately, and the installation based on the shorter of the two design lengths.

The selection of the maximum design load will depend on the configuration and strength of both the cable and immobilization system components. The selection of an appropriate maximum allowable deflection as a design criterion is more difficult since no data are currently available to indicate the effect of small amplitude oscillations on the

abrasion of cables. The design of immobilization systems based on maximum allowable deflection is presented in the anticipation that the information will become available

6.4.3 Selection of Maximum Design Load Conditions

An analysis of the failure mechanisms of cable/immobilization systems results in the following set of criteria:

(1) For $T_\Delta > \mu N T_B$, severe abrasion of the cable at the immobilization point may occur as the cable slides along the seafloor under the influence of the fastener pretension load.

(2) For $T_\Delta > \mu_s N T_B + \sigma_y^B (N\pi d_B^3/32h)$, plastic deformation of the seafloor fastener begins to occur, and

(3) For $T_I + T_\Delta > \sigma_y^c (\pi D^2/4)$, permanent deformation of the cable begins to occur with a resultant change in electrical and mechanical properties. Analysis of the effects of these critical loads suggests that the survivability of the cable is maximized if the design loads simultaneously satisfy Criteria 1 and 3:

$$\left.\begin{array}{l} T_\Delta < \mu_s N T_B \\ \text{and} \\ T_I + T_\Delta < \sigma_y^c \dfrac{\pi D^2}{4} \end{array}\right\} \quad \text{Condition 1} \qquad (6\text{-}23)$$

where σ_y^c is the yield stress of the cable. However, the state of the art of seafloor fasteners cannot guarantee the pretension load T_B will be maintained throughout

the design life of the system. If the pretension load should decrease to zero at some time after installation, then for the general case where $h/d_B > 1$, plastic deformation of the immobilization system will occur if Criteria 2* and 3 are not satisfied:

$$\left.\begin{aligned} &\dot{T}_\Delta < \sigma_y^B \frac{N \pi d_B^3}{32 h} \\ &\text{and} \\ &T_I + T_\Delta < \sigma_y^c \frac{\pi D^2}{4} \\ &\text{and} \\ &T_B = 0 \end{aligned}\right\} \quad \text{Condition 2} \qquad (6\text{-}24)$$

Since the state of the fastener pretension load throughout the life of the system cannot be predicted at the present time (1977), the survivability of the cable depends on satisfying both Conditions 1 and 2 throughout the life of the installation. This requirement can be satisfied by the following procedure:

(1) Define the parameter χ as the maximum tension (T_Δ) for Condition 2, divided by the tension resulting from the design of a system for Condition 1, which then loses its pretension load (T_B).

(2) Applying this criteria to Equation 6-17, χ can be expressed as:

*For $h/d < 1$, see equation for the maximum allowable load condition.

$$\chi = \frac{\sigma_y^B \pi d_B^3}{32 h \mu_s T_B} \left[1 + \frac{3 h^3 \pi^{3/2} F_H^* E_c^{3/2} D^3}{(24 \mu N T_B)^{3/2} N E_B I_B} \right]^{1/3} \quad (6\text{-}25)$$

(3) Both design load conditions will be satisfied if:

  (a) For $\chi < 1$, the system is designed for Condition 2.

  (b) For $\chi > 1$, the system is designed for Condition 1.

6.5 DESIGN OF CABLE IMMOBILIZATION SYSTEMS

Design equations that are valid for the conditions occurring at a specific installation site are developed from general Equations 6-17 through 6-22 by applying the following procedure:

(1) The appropriate design criteria (maximum allowable load or deflection) must be established, or an analysis for both cases carried out and the lesser of the two design span lengths used for the installation.

(2) The appropriate maximum allowable load condition must be determined by evaluating the function χ (Equation 6-25).

(3) The special transfer functions are established by substituting the appropriate load conditions into Equations 6-20 through 6-22.

(4) The cable pretension condition and specific transfer function are substituted into the general design equation to yield the required system spacing.

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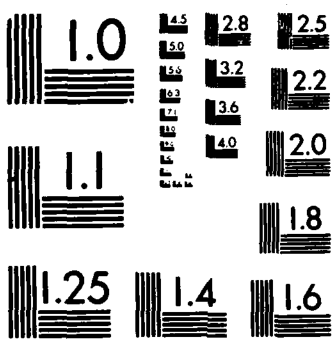

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The application of this procedure to the general design equations results in fourteen specific design equations. The development of these equations and their method of solution are presented in the following two sections.

6.5.1 Specific Design Equations for Maximum Allowable System Loads

The development of the maximum allowable load design equations is based on the solution of Equation 6-17 and the transfer function ψ_δ for the possible combinations of χ and T_I.

For the condition (χ>1), the maximum allowable loads for Condition 1 are substituted into Equation 6-20, and the specific design equation becomes:

$$\ell = \left(\frac{24 \hat{T}}{\pi F_H^{*2} E_c D^2} \right)^{1/2} \qquad (6\text{-}26)$$

where the term $\hat{T}$ has been substituted for $(T_I+T_\Delta)^2 T_\Delta$. The values of $\hat{T}$ are presented below for the four possible cases of T_I:

Case	T_I	$\hat{T}$
1	0	$(\mu_s N T_B)^3$
2	$> \sigma_y{}^c \frac{\pi D^2}{4} - \mu_s N T_B$	$\left(\frac{\sigma_y{}^c \pi D^2}{4}\right)^2 \left(\frac{\sigma_y{}^c \pi D^2}{4} - T_I\right)$
3	$< \sigma_y{}^c \frac{\pi D^2}{4} - \mu_s N T_B$	$(\mu_s N T_B + T_I)^2 (\mu_s N T_B)$
4	adjustable	$\left(\frac{\sigma_y{}^c \pi D^2}{4}\right)^2 (\mu N T_B)$

Cases 1 through 3 are for initial tensions that are fixed during laying of the cable or are due to the presence of suspensions (see Section 6.7) and cannot be changed prior to the installation of the immobilization system. Case 4 assumes that the cable tension is adjustable and set at an optimum value prior to the installation of the immobilization system. For this case, optimum spacing is achieved when the cable is pretensioned such that:

$$T_I = \sigma_y^c \frac{\pi D^2}{4} - \mu_s N T_B \qquad (6\text{-}27)$$

When the condition $\chi < 1$ exists, the specific design equations are obtained by substituting maximum allowable loads for Condition 2 into the transfer function for ψ_δ. The result is a third order equation for ℓ given by:

$$\ell^3 = \frac{24 \hat{T}}{\pi F_H^{*2} E_c D^2} \ell + \frac{2 \hat{T} h^3}{N F_H^{*2} E_B I_B} \qquad (6\text{-}28)$$

where the term $\hat{T}$ has again been substituted for the quantity $(T_I+T_\Delta)^2 T_\Delta$. The following table presents the values of $\hat{T}$ for Load Condition 2 and the four possible cases of T_I considered.

Case	T_I	$\hat{T}$
1	0	$\left(\sigma_y^B \frac{N \pi d_B^3}{32 h}\right)^3$
2	$> \sigma_y^c \frac{\pi D^2}{4} - \sigma_y^B \frac{N \pi d_B^3}{32 h}$	$\left(\sigma_y^c \frac{\pi D^2}{4}\right)^2 \left(\sigma_y^B \frac{N \pi d_B^3}{32 h} - T_I\right)$

Case	T_I	$\hat{T}$
3	$< \sigma_y^{c} \dfrac{\pi D^2}{4} - \sigma_y^{B} \dfrac{N \pi d_B^3}{32 h}$	$\left(\sigma_y^{B} \dfrac{\pi d_B^3}{32 h} + T_I\right)^2 \left(\sigma_y^{B} \dfrac{N \pi d_B^3}{32 h}\right)$
4	adjustable	$\left(\sigma_y^{c} \dfrac{\pi D^2}{4}\right)^2 \left(\sigma_y^{B} \dfrac{N \pi d_B^3}{32 h}\right)$

The method of solving Equation 6-28 for ℓ depends on the relative values of the tension and system parameters. If the quantities p, q, and τ are defined as follows:

$$p = \frac{8 \hat{T}}{\pi F_H^{*2} E_c D^2} \tag{6-29}$$

$$q = \frac{\hat{T} h^3}{N F_H^{*2} E_B I_B} \tag{6-30}$$

$$\tau = q^2 - p^3 = \frac{\hat{T}^2 h^6}{N^2 F_H^{*4} E_B^2 I_B^2} - \frac{512 \hat{T}^3}{\pi^3 F_H^{*6} E_c^3 D^6} \tag{6-31}$$

The value of τ calculated from Equation 6-31 will determine which of the three methods shown below should be used to solve for ℓ.

Method I - For $\tau > 0$ (one real root)

$$\ell = \left[q + (q^2 - p^3)^{1/2}\right]^{1/3} + \left[q - (q^2 - p^3)^{1/2}\right]^{1/3} \tag{6-32}$$

$$\ell = \left(\frac{\hat{T}\ h^3}{N\ F_H^{*2}\ E_B\ I_B} + \tau^{1/2}\right)^{1/3} + \left(\frac{\hat{T}\ h^3}{N\ F_H^{*2}\ E_B\ I_B} - \tau^{1/2}\right)^{1/3} \qquad (6\text{-}32a)$$

<u>Method II</u> - For $\tau = 0$

$$\ell_1 = 2\ q^{1/3} \qquad (6\text{-}33)$$

$$\ell_1 = 2\left(\frac{\hat{T}\ h^3}{N\ F_H^{*2}\ E_B\ I_B}\right)^{1/3} \qquad (6\text{-}33a)$$

$$\ell_2 = \ell_3 = -\ q^{1/3} \qquad (6\text{-}34)$$

$$\ell_2 = -\left(\frac{\hat{T}\ h^3}{N\ F_H^{*2}\ E_B\ I_B}\right)^{1/3} \qquad (6\text{-}34a)$$

<u>Method III</u> - For $\tau < 0$

$$\ell_1 = 2\ p^{1/2}\ \cos(u/3) \qquad (6\text{-}35)$$

$$\ell_1 = 2\left(\frac{8\ \hat{T}}{\pi\ F_H^{*2}\ E_c\ D^2}\right)^{1/2} \cos(u/3) \qquad (6\text{-}35a)$$

$$\ell_2 = 2\ p^{1/2}\ \cos(u/3 + 120°) \qquad (6\text{-}36)$$

$$\ell_2 = 2\left(\frac{8\,\hat{T}}{\pi\, F_H^{*2}\, E_c\, D^2}\right)^{1/2} \cos(u/3 + 120°) \qquad (6\text{-}36a)$$

$$\ell_3 = 2\, p^{1/2} \cos(u/3 + 240°) \qquad (6\text{-}37)$$

$$\ell_3 = 2\left(\frac{8\,\hat{T}}{\pi\, F_H^{*2}\, E_c\, D^2}\right)^{1/2} \cos(u/3 + 240°)$$

where: $u = \cos^{-1}(q/p^{3/2})$ (6-38)

$$= \cos^{-1}\left[\left(\frac{\pi}{8}\right)^{3/2}\left(\frac{h^3\, F_H^*\, E_c^{\,3/2}\, D^3}{N\,\hat{T}^{1/2}\, E_B\, I_B}\right)\right] \qquad (6\text{-}38a)$$

6.5.2 Specific Design Equations for Maximum Allowable Cable Deflection

The preceding section presented a method of designing a cable immobilization system so that mechanical failure of the system components would not occur. This approach does not place any restriction on the magnitude of the deflection of the midspan of the cable, which if large enough may result in abrasion failure. No data have been found that suggests a maximum "safe oscillating deflection" for various cables on the seafloor; however, the following discussion is presented in anticipation that this information may become available.

The equations for immobilization system spacing based on maximum allowable deflection are developed by solving Equation 6-18 and the transfer function $\psi_{T\Delta}$ for the possible combinations of χ and T_I. Although this design criterion places no restriction on the magnitude of

T_Δ, the development of the specific design equations assumes that allowable load conditions discussed in Section 6.4.3 are not exceeded. If the design spacing determined from the deflection analysis turns out to be greater than the spacing for the corresponding allowable load analysis, the assumption is not valid and the installation design should be based on the equations presented in Section 6.5.1.

For the condition $\chi > 1$, the allowable deflection design equation becomes:

$$\ell^4 = \frac{2\, T_I\, \delta}{F_H^*}\, \ell^2 + \frac{\pi\, \delta^3\, E_c\, D^2}{3\, F_H^*} \tag{6-39}$$

The exact solution of this equation depends on the value of T_I as shown below.

<u>Case 1</u> - For $T_I = 0$

$$\ell = \left(\frac{\pi\, \delta^3\, E_c\, D^2}{3\, F_H^*} \right)^{1/4} \tag{6-40}$$

<u>Case 2</u> - For $T_I > 0$ and nonadjustable

$$\ell = \left[\frac{T_I\, \delta + \left({T_I}^2\, \delta^2 + \frac{\pi}{3}\, F_H^*\, \delta^3\, E_c\, D^2 \right)^{1/2}}{F_H^*} \right]^{1/2} \tag{6-41}$$

<u>Case 3</u> - For $T_I > 0$ and adjustable

The optimum value of T_I is obtained by solving Equation 6-19 for the condition $T_I + T_\Delta = \sigma_y{}^c(\pi D/4)$. The optimum value of T_I is thus found to be:

$$T_I = \sigma_y{}^c \frac{\pi D^2}{4} - \frac{\delta F_H^* E_c}{3 \sigma_y{}^c} \tag{6-42}$$

and the design spacing (L_c) is found by substituting this value into Equation 6-41 and solving for $\ell = L_c/2$.

For the condition $\chi < 1$, Equation 6-18 becomes the quartic equation:

$$\ell^4 + \left(\frac{\pi h^3 E_c D^2}{12 N E_B I_B}\right)\ell^3 - \left(\frac{2 T_I \delta}{F_H^*}\right)\ell^2 - \left(\frac{\pi \delta T_I h^3 E_c D^2}{6 F_H^* N E_B I_B}\right)\ell - \left(\frac{\pi \delta^3 E_c D^2}{3 F_H^*}\right) = 0 \tag{6-43}$$

Substituting the possible conditions for T_I results in the following three solutions for ℓ.

<u>Case 1</u> - For $T_I = 0$

$$\ell^4 + \left(\frac{\pi h^3 E_c D^2}{12 N E_B I_B}\right)\ell^3 - \left(\frac{\pi E_c D^2 \delta^3}{3 F_H^*}\right) = 0 \tag{6-44}$$

The solution of this equation is most easily obtained by the iteration procedure described below.

Rewrite Equation 6-44 in terms of ℓ and δ/ℓ,

$$\left(\frac{3\ F_H^*}{\pi\ E_c\ D^2}\right)\ell + \left(\frac{F_H^*\ h^3}{4\ E_B\ I_B}\right) = \left(\frac{\delta}{\ell}\right)^3 \tag{6-45}$$

Let $A = \dfrac{3\ F_H^*}{\pi\ E_c\ D^2}$ and $B = \dfrac{F_H^*\ h^3}{4\ E_B\ I_B}$

Then Equation 6-45 may be written in the iterative form:

$$\ell_{i+1} = \left[\frac{\left(\frac{\delta^3}{A}\right)}{\ell_i + \frac{B}{A}}\right]^{1/3} \tag{6-46}$$

where $\ell_i = \ell_o$ may be any arbitrary value greater than zero and is usually specified as unity. In most cases this iteration converges rapidly with $\ell_3 \approx \ell$, thereby giving the design length well within the accuracy that can be achieved during an actual installation.

<u>Cases 2 and 3</u> - For $T_I > 0$

The general solution for Equation 6-43 is complex and extremely time consuming. Many programmable calculators and minicomputers have prepared programs that solve for the roots of fourth order equations and their use is recommended. For the case where T_I is adjustable, the optimum value calculated from Equation 6-19 is found to be

$$T_I = \sigma_y^c \frac{\pi D^2}{4} - \frac{\delta^2}{\dfrac{3\,\delta\,\sigma_y^c}{E_c\,F_H^*} + \left(\dfrac{\delta\,\sigma_y^c\,\pi\,D^2\,h^6}{8\,F_H^*\,N^2\,E_B^2\,I_B^2}\right)^{1/2}} \qquad (6\text{-}47)$$

6.5.3 Application of Cable Immobilization System Design Equations

The application of Equations 6-17 through 6-47 to the design of an actual immobilization system will depend upon the objective of the design process. For the general situation where the components of the immobilization system are defined, these equations are used in the form presented in Sections 6.5.1 and 6.5.2 to determine the maximum allowable spacing between the immobilization points. However, when the spacing is initially specified, either due to limitations in the number of points that can be installed within the weather window or the distance between competent rock outcrops, the design equations may be used to specify the strength, pretension, configuration, and number of seafloor fasteners at each immobilization point.

6.6 EFFECTIVE MODULUS OF ELASTICITY OF OCEAN CABLES

The development of the design theory for immobilization of ocean cables is based upon the condition of quilibrium achieved when the externally applied hydrodynamic forces are balanced by the internal strain induced forces in the cable. From Equation 6-5, this internal force is found to be a function of the immobilization system geometry and modulus of elasticity of the cable. The equations developed in the previous sections considered the cable to be composed of a homogeneous material with modulus of elasticity (E_c). Since actual ocean cables are composite structures, the "effective modulus of elasticity" may be determined from the equation:

$$E_c = \frac{4 \sum_{i=1}^{n} E_i A_i}{\pi D^2} \tag{6-48}$$

where E_c = effective modulus of elasticity of the cable

E_i = modulus of elasticity of the i^{th} circumferential component of the cable

A_i = cross-sectional area of the i^{th} circumferential component of the cable

D = diameter of the cable

Table 6-2 provides a list of the mechanical properties of the most common materials used in the fabrication of ocean cables. Ocean cables used in the nearshore zone are almost always fabricated with one or more layers of outer armor wires, which are provided for both abrasion protection and the transmission of axial forces (see Section 2.6.3 and 4.2.1). The spiral configuration of these wires cause them to react similar to an open-coiled helical spring when subjected to axial cable loads.

Table 6-2. Mechanical Properties of Materials Most Commonly Used in Ocean Cables

Material	Modulus of Elasticity, E (psi)	Poisson's Ratio, μ_p	Yield Stress, σ_y (psi x 10^3)	Ultimate Stress, σ_u (psi x 10^3)
Steel (armor wire)	30×10^6	0.28-0.29	30-40	50-65
Steel, high strength	30×10^6	0.28-0.29	40-80	65-90
Copper	15.6×10^6	0.355	5	32
Lead				
Cast iron	$13\text{-}21 \times 10^6$	0.21-0.29	8-40	18-60
Stainless steel	27.6×10^6	0.30	30-35	85-95
Polyethylene				
High density	$0.8\text{-}1.5 \times 10^5$			3-5.5
Low density	$0.17\text{-}0.35 \times 10^5$			1-2.3

When an axial load P is applied to an open-coiled helical spring that is restrained from rotating, the theoretical deflection of the spring is given by the equation:*

$$\delta_w = P R^2 s_w \left\{ \frac{\cos^2 \alpha_w}{G I_p} + \frac{\sin^2 \alpha_w}{E I} - \frac{\sin^2 \alpha_w \cos^2 \alpha_w \left[\left(\frac{1}{G I_p} \right)^2 - \left(\frac{1}{E I} \right)^2 \right]}{\left(\frac{\sin^2 \alpha_w}{G I_p} + \frac{\cos^2 \alpha_w}{E I} \right)} \right\} \qquad (6\text{-}49)$$

where α_w = helical angle of armor wire = $\tan^{-1}(\ell_w/2\pi R)$

P = axial load

R = radius of spring

s_w = length of spring wire

ℓ_w = length of spring

G = modulus of rigidity = $E/2(1+\mu_p)$

E = modulus of elasticity

I = moment of inertia of cross section of spring wire

I_p = polar moment of inertia of cross section of spring wire

μ_p = Poisson's ratio

Substituting this force deflection equation into the equation:

$$E A = \frac{P \ell}{\delta}$$

*Derived from Timoshenko (1956).

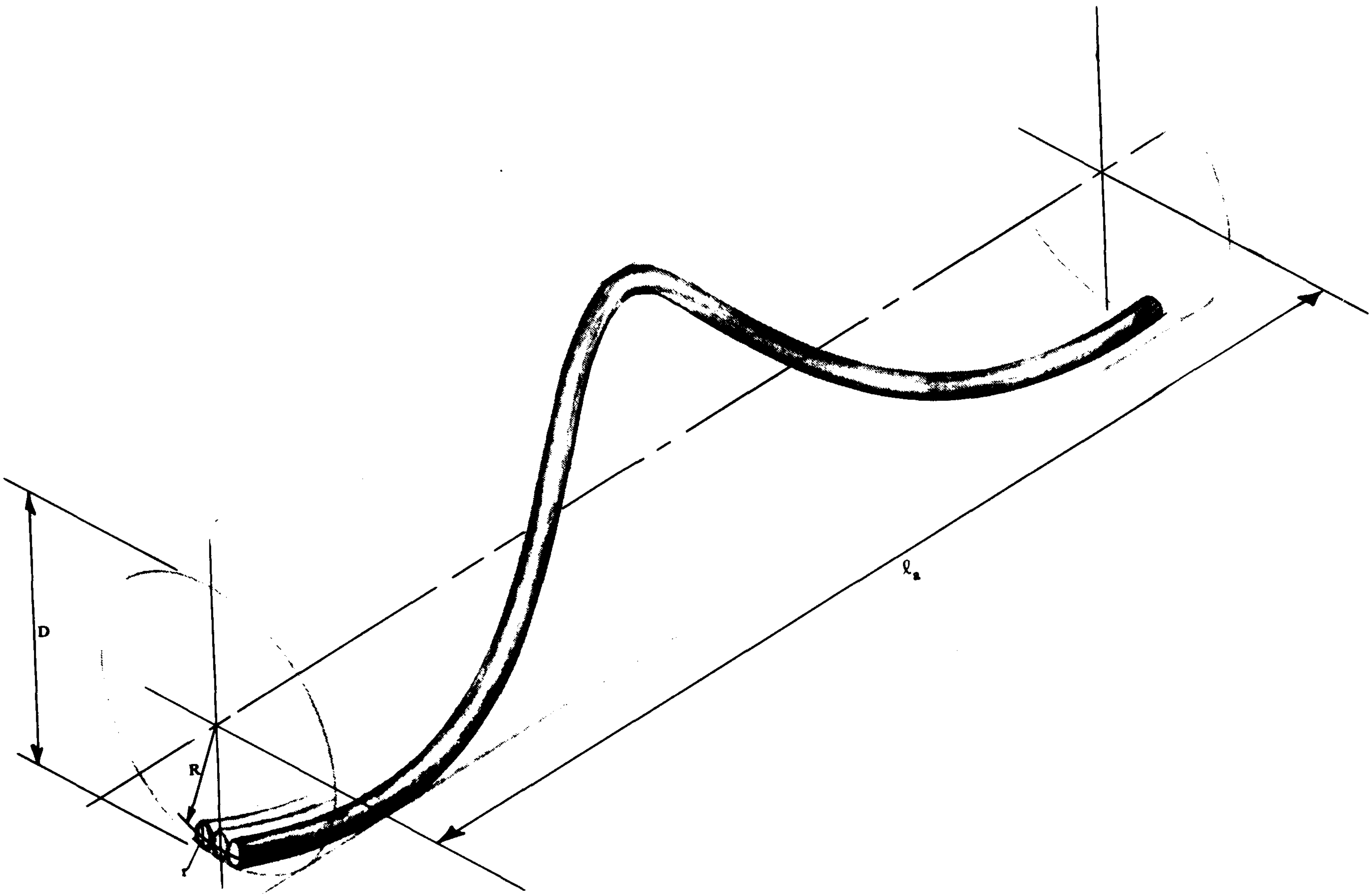

Figure 6-10. Helical configuration of cable armor wire.

and evaluating the moments of inertia in terms of the geometry of a cable (Figure 6-10), the elastic modulus - area coefficient ($E_i A_i$) for each layer or armor can be obtained from:

$$E_i\, A_i = \frac{\pi\, n_w E\, r^4 \sin \alpha_w (\gamma_p \sin^2\alpha_w + \cos^2\alpha_w)}{4R^2(\gamma_p \sin^4\alpha_w + 2 \sin^2\alpha_w \cos^2\alpha_w + \gamma_p \cos^4\alpha_w)} \qquad (6\text{-}50)$$

where α_w = $\tan^{-1}(\ell_a/2\pi R)$

γ_p = $(1 + \mu_p)$

n_w = number of armor wires per layer

E = modulus of elasticity of armor wire material

r = radius of individual armor wire

R = mean radius of armor layer

ℓ_a = length of armor lay

μ_p = Poisson's ratio

The modulus-area coefficient for each layer of armor must be calculated separately using Equation 6-50. These values plus the modulus-area coefficients for each component of the core of the cable (obtained from the cable geometry and Table 6-2) are then substituted into Equation 6-48 to obtain the effective modulus of elasticity of the composite cable.

6.7 STABILIZATION OF CABLES UNDER THE INFLUENCE OF SUSPENSION AND HYDRODYNAMICALLY INDUCED LOADS

6.7.1 Suspended Portion of Cable

The effect of suspension-induced loads will not normally be included in the intitial stabilization design calculations since the existence and configuration of suspensions are difficult if not impossible to

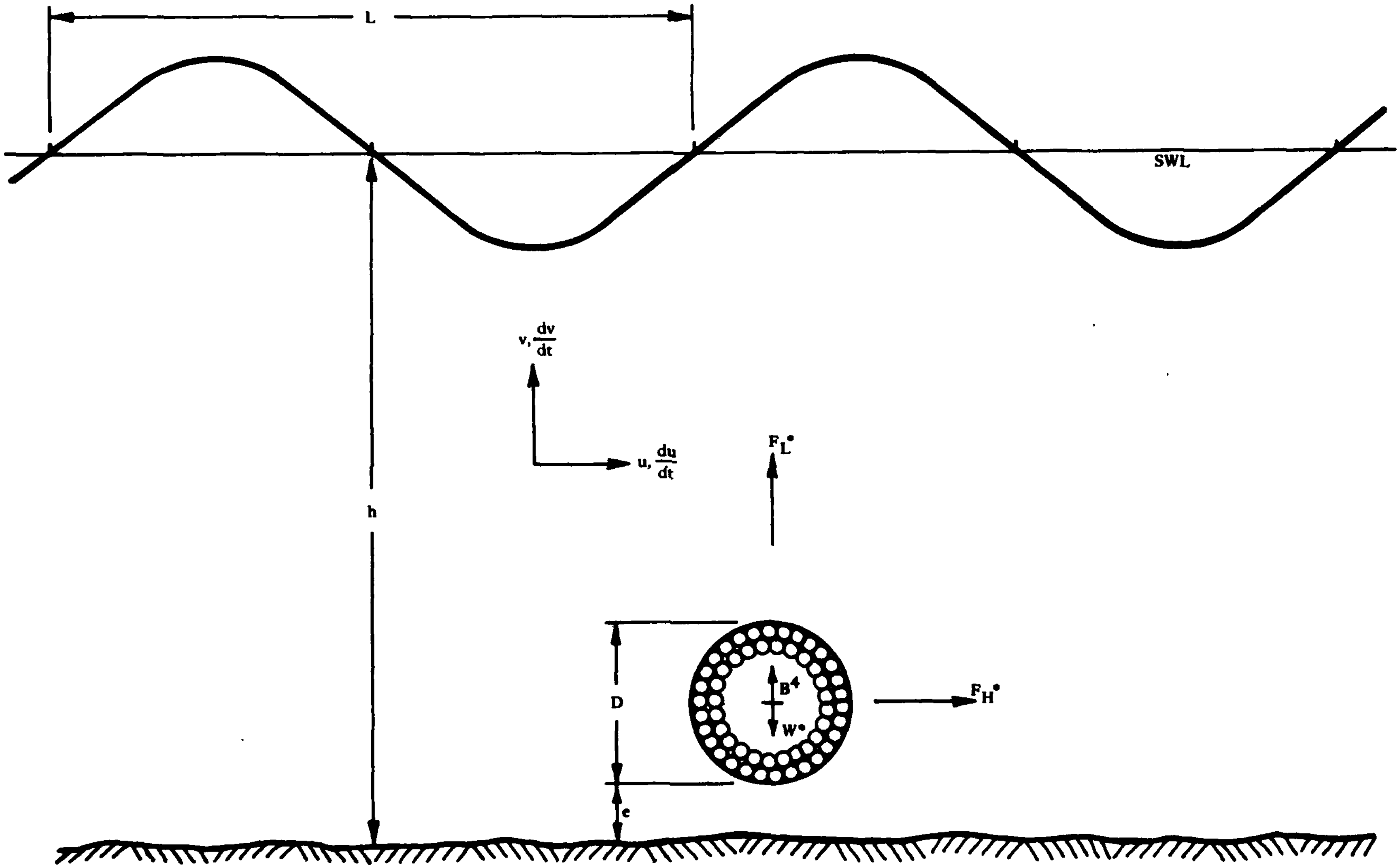

Figure 6-11. Hydrodynamic forces acting on suspended ocean cable.

determine until after the cable is in-place on the seafloor. The discussion presented in this section, therefore, is intended primarily for on-site modification of stabilization system design.

When a portion of a cable is suspended above the seafloor, no friction forces act on that length and, using the criteria established in Section 6.2, the suspension is unstable when subjected to hydrodynamic loads. Figure 6-11 shows the forces acting perpendicular to the axis of the cable. For an e/D ratio greater than 2, the net lift force is approximately zero, and the horizontal forces acting on a unit length of cable are given by:

$$F_H^* = \frac{1}{2} C_D \rho D u^2 + \frac{\pi}{4} C_I \rho D^2 \frac{du}{dt} \tag{6-51}$$

where $C_D \approx 2.5\ C_D'$ (from Section 5.4.1). The submerged weight per unit length of cable (W_s^*) is given by:

$$W_s^* = W^* - B^* \tag{6-1}$$

$$W_s^* = \frac{\pi}{4}\left[\sum \rho_i (D_{o_i}{}^2 - D_{I_i}{}^2) - D^2 \rho_w\right] \tag{6-52}$$

where
W_s^* = submerged weight per unit length
W^* = weight (in air) per unit length
B^* = buoyant force per unit length
ρ_i = density of i^{th} circumferential component of cable
D_{o_i} = outside diameter of i^{th} circumferential component of cable
D_{I_i} = inside diameter of i^{th} circumferential component of cable
D = diameter of cable
ρ_w = density of seawater

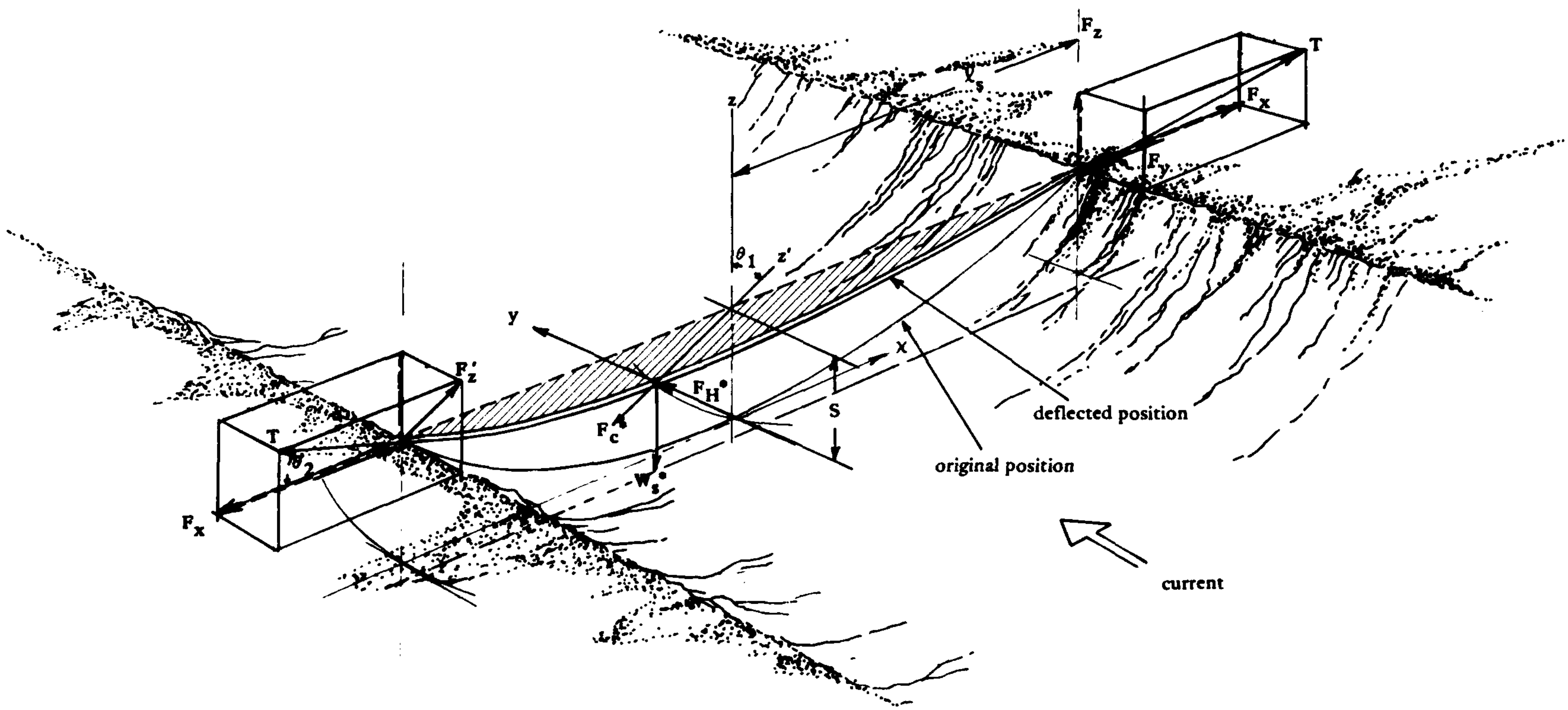

Figure 6-12. Coordinate system for suspended ocean cable.

These unit forces can be resolved, using vector algebra, to give the equivalent single force (F_c^*) acting at an angle (θ_1) to the vertical axis such that:

$$F_c^* = \left(F_H^{*2} + W_s^{*2}\right)^{1/2} \tag{6-53}$$

$$\theta_1 = \tan^{-1}\left(\frac{F_H^*}{W_s^*}\right) \tag{6-54}$$

For the purpose of analysis, the torsional resistance of the cable is neglected, and the suspension is considered to rotate as a rigid body through an angle θ_1 as shown in Figure 6-12. The internal forces in the cable, determined from a catenary analysis in the xz' plane, are given by:

$$T_{max} = F_c^*\left(\frac{\ell_c^{\,2} - S^2}{2\,S}\right)\cosh\left(\frac{2\,\ell_s\,S}{\ell_c^{\,2} - S^2}\right) \tag{6-55}$$

$$F_x = T_o = F_c^*\left(\frac{\ell_c^{\,2} - S^2}{2\,S}\right) \tag{6-56}$$

$$F_{z'} = F_c^*\,\ell_s \tag{6-57}$$

and the angle between the cable and x axis (in the xz' plane) is given by

$$\theta_2 = \cos^{-1}\left(\frac{T_o}{T_{max}}\right) = \cos^{-1}\left[\cosh\left(\frac{2\,\ell_s\,S}{\ell_c^{\,2} - S^2}\right)\right]^{-1} \tag{6-58}$$

If the maximum tension force is resolved into orthogonal forces in the xyz coordinate system, then

$$F_x = T_o = F_c^* \left(\frac{\ell_c^{\,2} - S^2}{2\,S} \right) = T_{max} \sin \theta \qquad (6\text{-}59)$$

$$F_y = F_H^* \, \ell_s = T_{max} \sin \theta_2 \sin \theta_1 \qquad (6\text{-}60)$$

$$F_z = W_s^* \, \ell_s = T_{max} \sin \theta_2 \cos \theta_1 \qquad (6\text{-}61)$$

6.7.2 Bottom-Resting Portion of Cable

When a suspended cable contacts the seafloor at the support point, friction forces must again be considered. For a cable approaching the support point at an angle θ_2 with respect to the χ axis, the tension in the cable is reduced to a value T' due to friction, as shown in Figure 6-13. The value of T' at the support point is given by the equation:

$$T' = \frac{T_{max}}{e^{\mu_s \theta_2}} \qquad (6\text{-}62)$$

where e = constant, ≈ 2.718

θ_2 = angle expressed in radians

The normal and friction forces are:

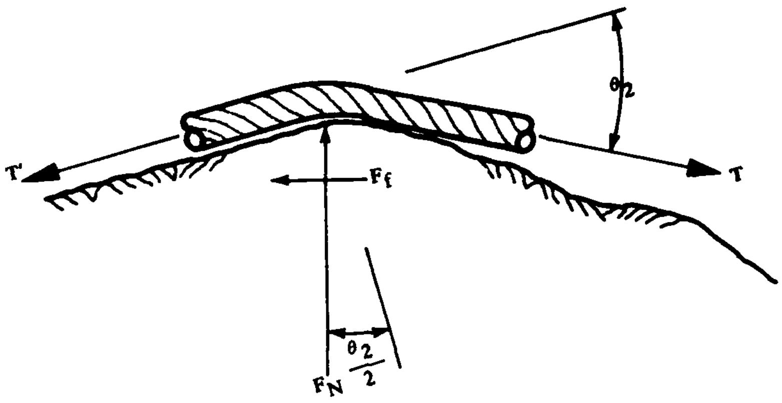

Figure 6-13. Support point configuration of a suspended ocean cable.

$$F_N = \frac{F_c^* \left(\frac{\ell_c^2 - S^2}{2\,S} \right)}{\left[\cos\left(\frac{\theta_2}{2}\right) + \mu \sin\left(\frac{\theta_2}{2}\right) \right]} \qquad (6\text{-}63)$$

$$F_f = \mu\, F_N \qquad (6\text{-}64)$$

Moving away from the support point a distance ℓ', the cable tension is reduced by friction by an amount:

$$\Delta T = \mu_s\, W_s^*\, \ell' \qquad (6\text{-}65)$$

Figure 6-14 shows the internal tension of a cable as a function of the geometry of a suspended cable. The distance ℓ'_{cr}, at which the internal tension reduces to zero, defines the range of influence of a suspension on the bottom-resting portion of the cable. From Figure 6-14 the range of influence is given by:

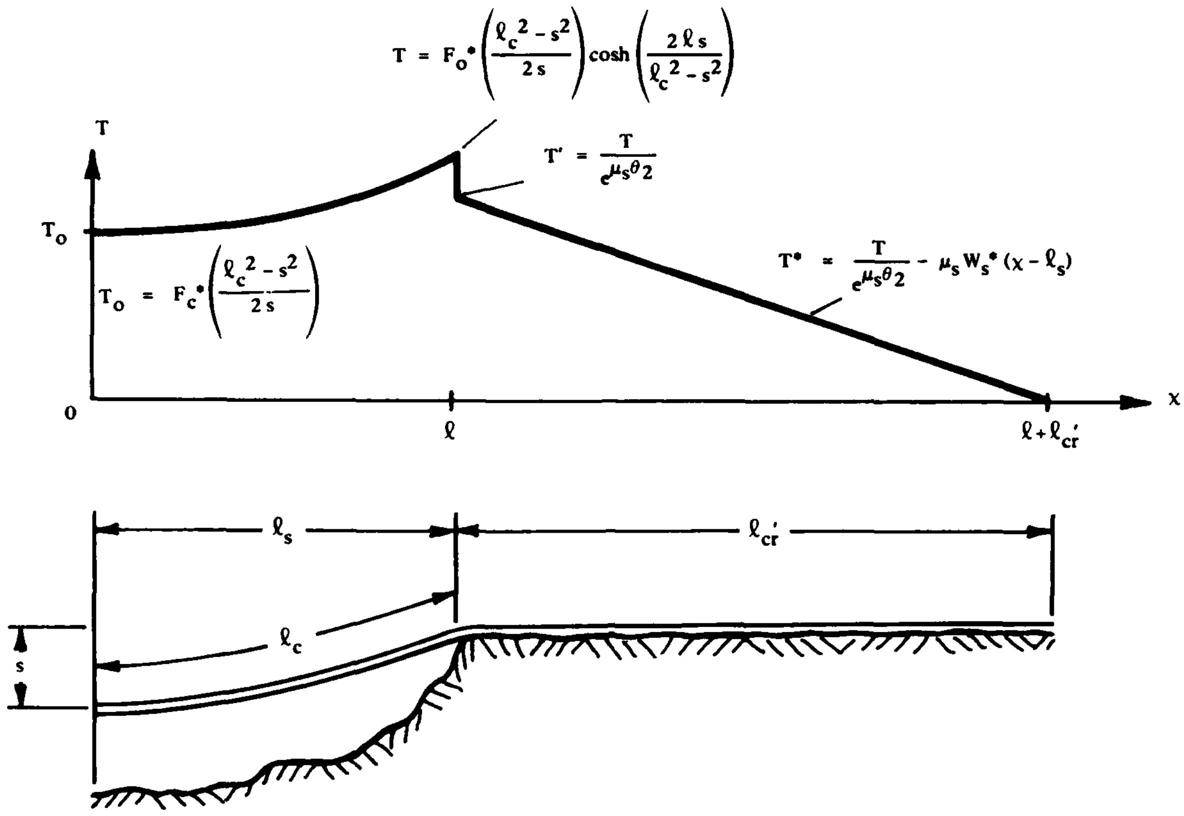

Figure 6-14. Internal force distribution resulting from suspension and friction forces.

$$\ell'_{cr} = \frac{F_c^*\left(\frac{\ell_c^2 - s^2}{2s}\right)\cosh\left(\frac{2\ell_s s}{\ell_c^2 - s^2}\right)}{\mu_s W_s^* e^{\mu_s\theta_2}} \tag{6-66}$$

6.7.3 Modification of Immobilization System Design

The immobilization of the support points of a cable suspension results in an indeterminate system of forces. A conservative design solution is obtained, however, if the friction resistance is neglected in the direction of the y axis. Utilizing this assumption, the force applied to the seafloor fastener at the immobilization point is given by:

$$F_B = \left(T'^2 + F_y^2\right)^{1/2}$$

$$= \left\{ \left[\frac{F_c^* \left(\frac{\ell_c^2 - S^2}{2\,S}\right) \cosh\left(\frac{2\,\ell_s\,S}{\ell_c^2 - S^2}\right)}{e^{\mu_s \theta_2}} \right]^2 + F_H^{*2}\,\ell_s^2 \right\}^{1/2} \qquad (6\text{-}67)$$

For immobilization points located at a distance less than ℓ'_{cr} from the support point, the effect of the internal tension which is given by:

$$T_I = \frac{T}{e^{\mu_s \theta_2}} - \mu_s\, W_s^*\, \ell' \qquad (6\text{-}68)$$

must be taken into account (as discussed in Section 6.5) when designing the immobilization system.

6.8 SUMMARY

This chapter has provided cable protection system design equations for both stabilization (mass anchors) and immobilization (tie-downs) based on anticipated maximum hydrodynamic loads (presented in Chapter 5). The following chapter (Chapter 7) presents a systematic procedure for using these equations and those presented in Chapter 5 to develop the design of an actual cable protection system.

Chapter 7

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DESIGN PROCEDURE

7.1 INTRODUCTION

Very few attempts have been made to establish formal procedures for designing stabilization or immobilization systems for ocean cables. The only documented design procedure found in the literature was developed by Cullison (1975). This chapter presents a systematic procedure for establishing and analyzing the hydrodynamic loading on an ocean cable system and subsequently designing a stabilization and/or immobilization system to allow the cable to resist these loads. The procedure presented in this chapter is based on the work by Cullison and on the information contained in Chapters 3 through 6 of this handbook.

For clarity, the design process has been presented in seven phases: (1) development of a model of the nearshore zone, (2) shoaling and refraction analysis, (3) breaking wave analysis, (4) hydrodynamic force analysis, (5) stability analysis, (6) immobilization system design, and (7) economic analysis. A summary section has also been included with a flow chart showing the step-by-step process of reaching an optimum design for a particular site and cable system requirements.

7.2 DEVELOPING A MODEL OF THE NEARSHORE ZONE

An analytical design of a nearshore cable stabilizaton/immobilization system must start with the definition of the environment in the nearshore region. This definition is most easily accomplished by developing a model that can then be used as input to the design process. Six basic parameters must be identified or defined for this model: (1) topography of the seafloor and beach area, (2) probable cable path, (3) design wave parameters, (4) appropriate significant wave height, (5) initial analysis point along the cable path, and (6) water depth and wave length at the analysis points.

7.2.1 Topographic Model

A topographic model of the seafloor can be developed from data obtained either from coast and geodetic survey charts or the results of the site survey as discussed in Section 2.3.2. This model should show the topographic contours and shore line for at least a half mile on either side of the proposed cable landing point to assure installation variations would not create a critical situation. The feasibility of some mass anchor stabilization techniques (shore-applied split-pipe and oil field pipe) will also depend on the topography of the beach area; this information should be included on the topographic model for reference purposes.

If the site survey reveals variations in seafloor material, the regions of various material should be identified to assist in determining which of the feasible techniques are applicable along various portions of the cable route.

7.2.2 Identifying the Probable Cable Path

The probable cable path is next located on the topographic chart. In the case where the cable has been installed prior to the stabilization/immobilization system design, an exact cable path can be plotted. Often,

however, this design process will take place during the cable installation planning phase, and, therefore, the exact location of the cable after installation cannot be guaranteed. The identification of a probable path will depend on the following factors:

(1) Mission requirements that will dictate the general landing point and direction of the cable to sea.

(2) Topographic characteristics of the seafloor which may suggest a best route or direction to avoid the possibility of cable suspension or unstable seafloor materials.

(3) Existing facilities, such as previously installed cables and pipelines that should be avoided if possible. If this is impossible, the new cable path should cross the existing seafloor installations as close to right angles as possible.

(4) Assessment of the capability to install the cable along the specified path. This includes such factors as the type of ships to be used, their ability to maintain position during the landing operation, and the length of time the ship must remain on station. The possibility of large catenaries developing due to wind or current before the cable is positioned on the seafloor, and the capability of repositioning the cable once it is installed must also be considered.

(5) Hydrodynamic factors, such as the direction of maximum current and waves that can influence the selection of a cable path. Since the magnitude of the hydrodynamic forces is influenced by the angle of incidence of the impinging water particles, the combined effects of current and surge can sometimes be used to indicate an optimum cable path along which the maximum water particle velocity will always be parallel to the cable.

The evaluation of these five factors can be used to identify both a best and worst case cable path. For critical installations, a conservative design is achieved if the cable installation is based on a best path and the stabilization/immobilization system design is based on a worst case path.

7.2.3 Selection of the Storm Wave Parameters

The selection of wave height, period, and direction will have a large economic impact on the installation of the system. Specifying unrealistically large wave parameters will result in an unwarranted expenditure of funds, while too moderate a wave environment will most often result in premature failure of the cable system. Since the accurate prediction of the maximum wave environment during the life of the cable is almost impossible, the selection of the design wave must be based on establishing a reasonable probability of occurrence, or, more accurately, using a design wave such that the probability of exceeding the design hydrodynamic forces is acceptably small.

If the quantity T_R is defined as the average time of return (in years) of a particular storm wave environment and n is the required number of years that the cable system must remain operational, then the probability P_n that the wave environment will exceed that of the design wave during the life of the installation is given by:

$$P_n = 1 - \left(1 - \frac{1}{T_R}\right)^n \tag{7-1}$$

Rearranging this equation and solving for T_R as a function of P_n and n,

$$T_R = \frac{1}{1 - (1 - P_n)^{1/n}} \tag{7-2}$$

The family of curves for various installation life requirements that satisfy Equation 7-2 have been plotted in Figure 7-1. Also plotted (dash lines) are the probability curves for T_R equal to various multiples of n, it is interesting to note that for the practice of selecting the time of return of the design wave equal to the installation life requirement, the probability of exceeding the hydrodynamic design loads is greater than 62%. In order to reduce the probability of exceeding the design loads to a 10% level, the return time must be at least 10 times the installation life requirements. This means that for a critical installation with a 20-year life requirement, the stabilization/immobilization system should probably be designed to resist the hydrodynamic forces produced by a 200-year storm.

Once the time of return of the design storm is obtained, the wave parameters (height, period, and direction) associated with these storm conditions must be obtained. The most desirable sources of this type of information are discussed in Sections 2.3.6 and include hydrographic reports prepared for the specific area of interest, consultation with local meterological agencies or universities, or, if the time of return is not exceptionally long, data from personnel involved with the local marine industry. If these resources are not available at the site of interest, the design wave parameters can be obtained using forecasting techniques.

Numerous wave parameter forecasting techniques have been developed based on both theoretical and empirical data. The Sverdrup-Munk-Bretschneider (SMB) method is the most convenient of these techniques when a limited amount of data and time are available (Army CERC, 1973). In order to implement this technique, data must be obtained on maximum wind speed occurring during the life of the installation, direction of the wind, the fetch length (unobstructed distance over which the wind blows), and the duration of the maximum wind.

The maximum wind speed U produced by a storm having a return period of T_R years was found by Thom (1961) to be reasonably well described by the relationship

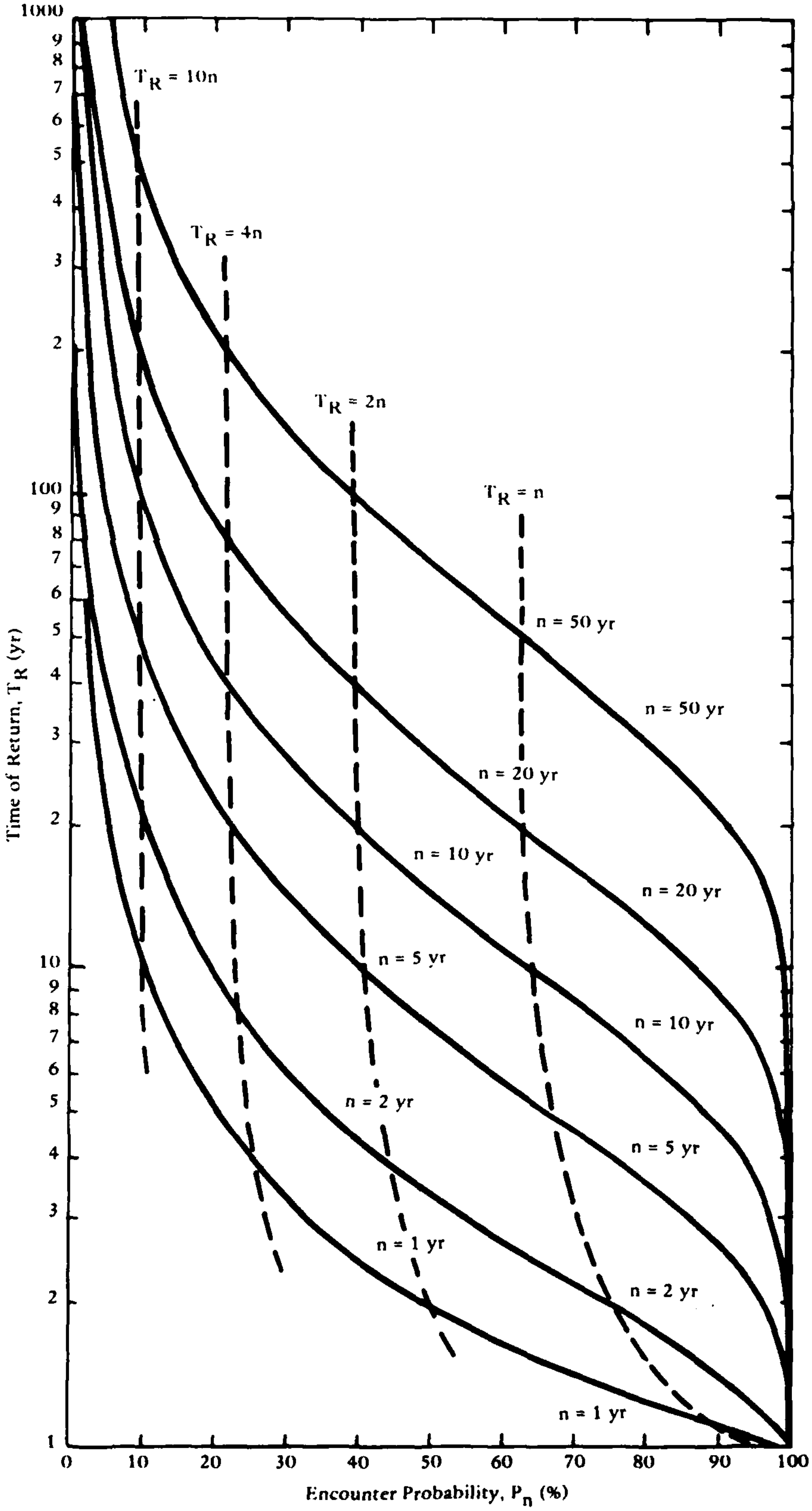


Figure 7-1. Encounter probability versus time of return for various installation life requirements.

$$\left(\frac{U}{\beta}\right)^{-\gamma} = \ln\left(\frac{T_R}{T_R - 1}\right) \quad (7\text{-}3)$$

where β and γ are parameters characterizing the wind speed probability distribution. Equation 7-3 conforms to a Fischer-Tippett Type II probability distribution. When plotted on logarithmic probability paper developed for this type of distribution, wind-speed-return-period data will plot as a straight line. Figure 7-2 is a grid developed for this type of distribution; it has intentionally been presented without any data points so that it may be used for design calculations.

To determine the design wind speed produced by a storm with a return period of T_R years, wind data associated with two or more return periods are plotted on Figure 7-2. The design wind speed is obtained from the intersection of the straight line drawn through these data points and coordinate for the desired value of T_R. An example of the use of this prediction technique is presented in Myers et al. (1969).

Theoretical techniques for determining the expected fetch, duration, and direction of the wind do not exist, and selection of appropriate values for these parameters must be obtained from knowledge of local site conditions and historical data. Procedures for gathering or estimating this information have been thoroughly discussed by Wiegel (1964), Army CERC (1973), and Myers et al. (1969).

The Sverdrup-Munk-Bretschneider wave parameter forecasting curves are presented in Figure 7-3. Using the maximum design wind speed value obtained from Figure 7-2, Figure 7-3 is entered with the value of U (using the scale on the left if U is in knots or the scale on the right if U is in statute miles per hour). This U line is then followed from the left side of the graph across to its intersection with the fetch length line or duration line, whichever comes first. The significant wave height and period are obtained by interpolating the values from the appropriate parametric curves on either side of the intersection point. Procedures for forecasting wave parameters when the wind velocity is variable are discussed in detail in the Shore Protection Manual, Vol I (Army CERC, 1973).

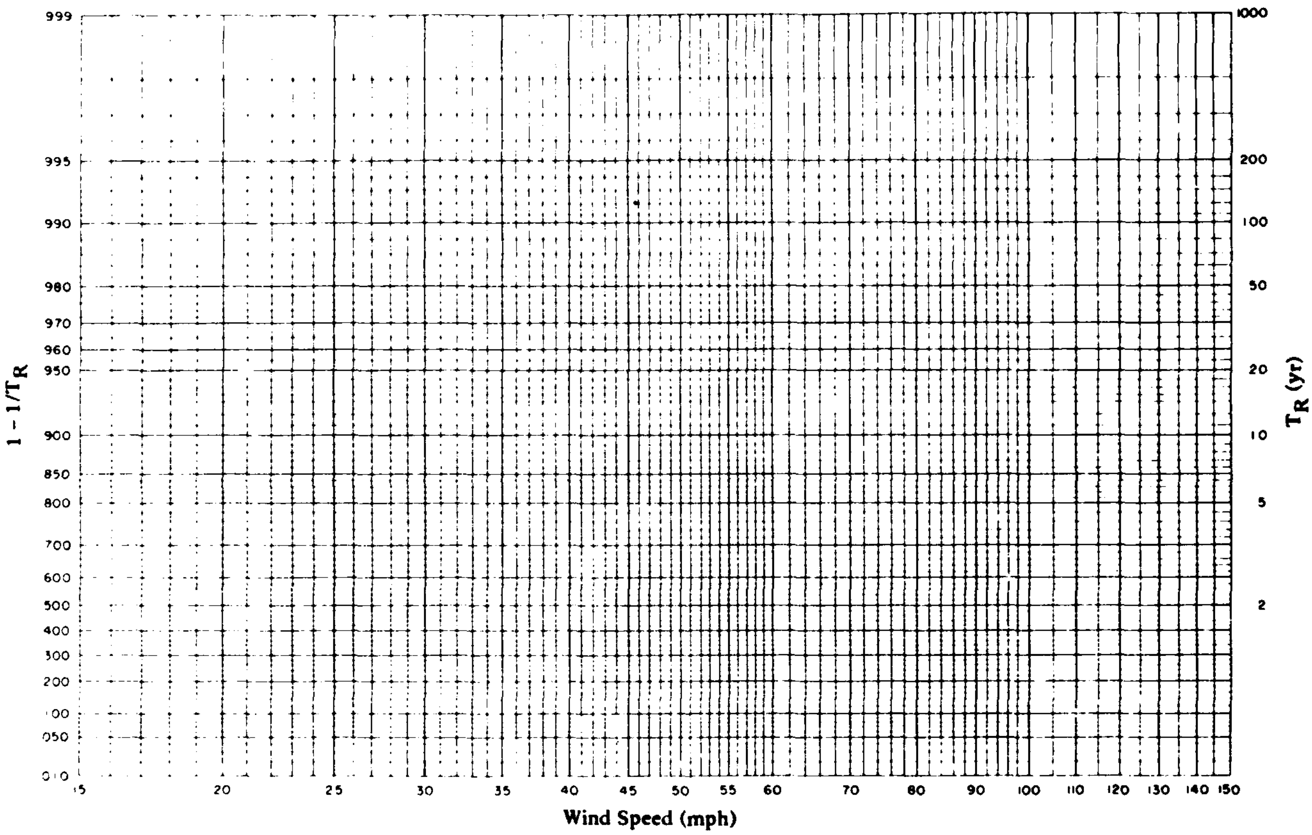


Figure 7-2. Logarithmic probability paper for maximum wind speed (Fischer-Tippett Type II distribution).

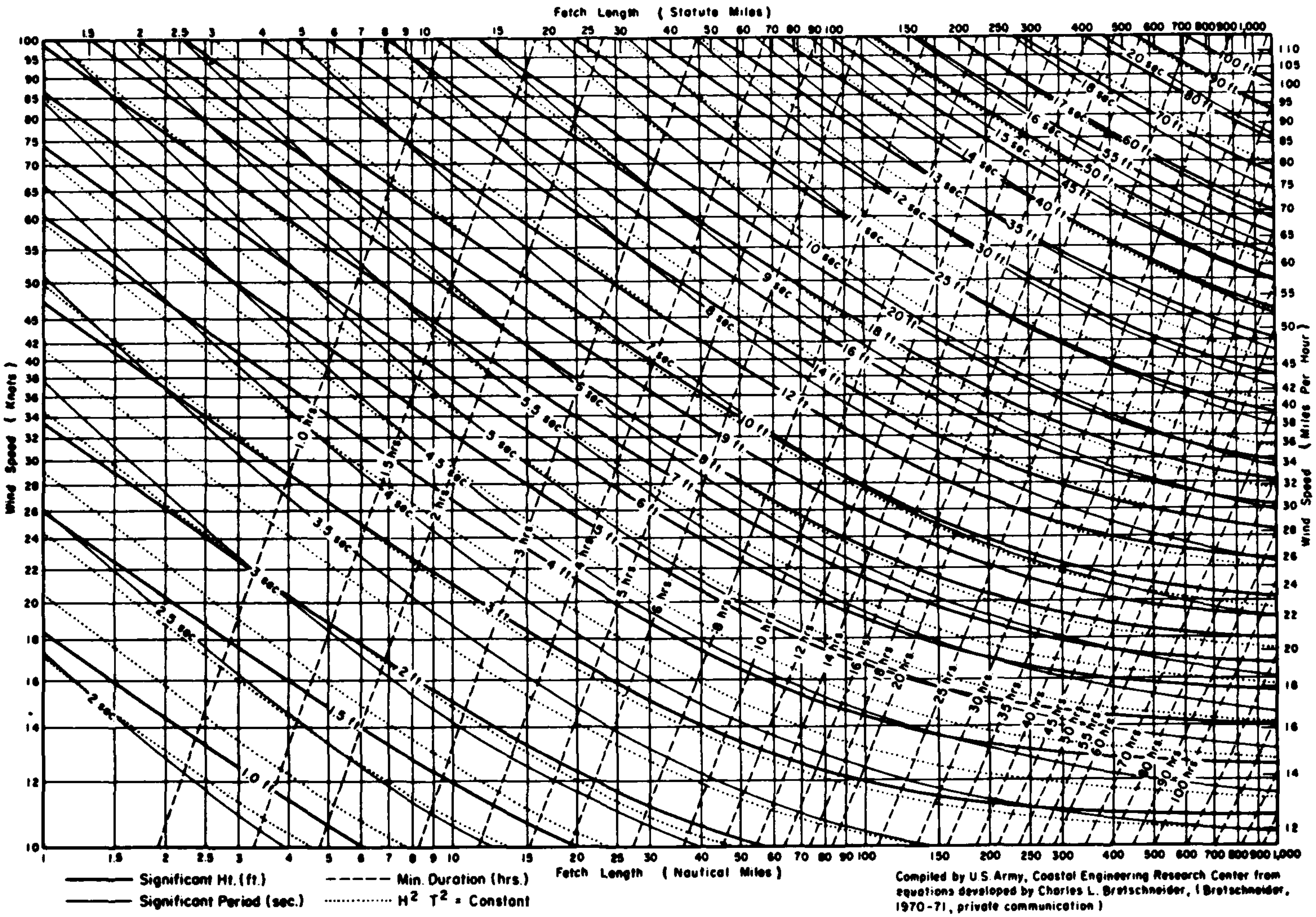


Figure 7-3. Forecasting curves for wave height and period (SMB method).

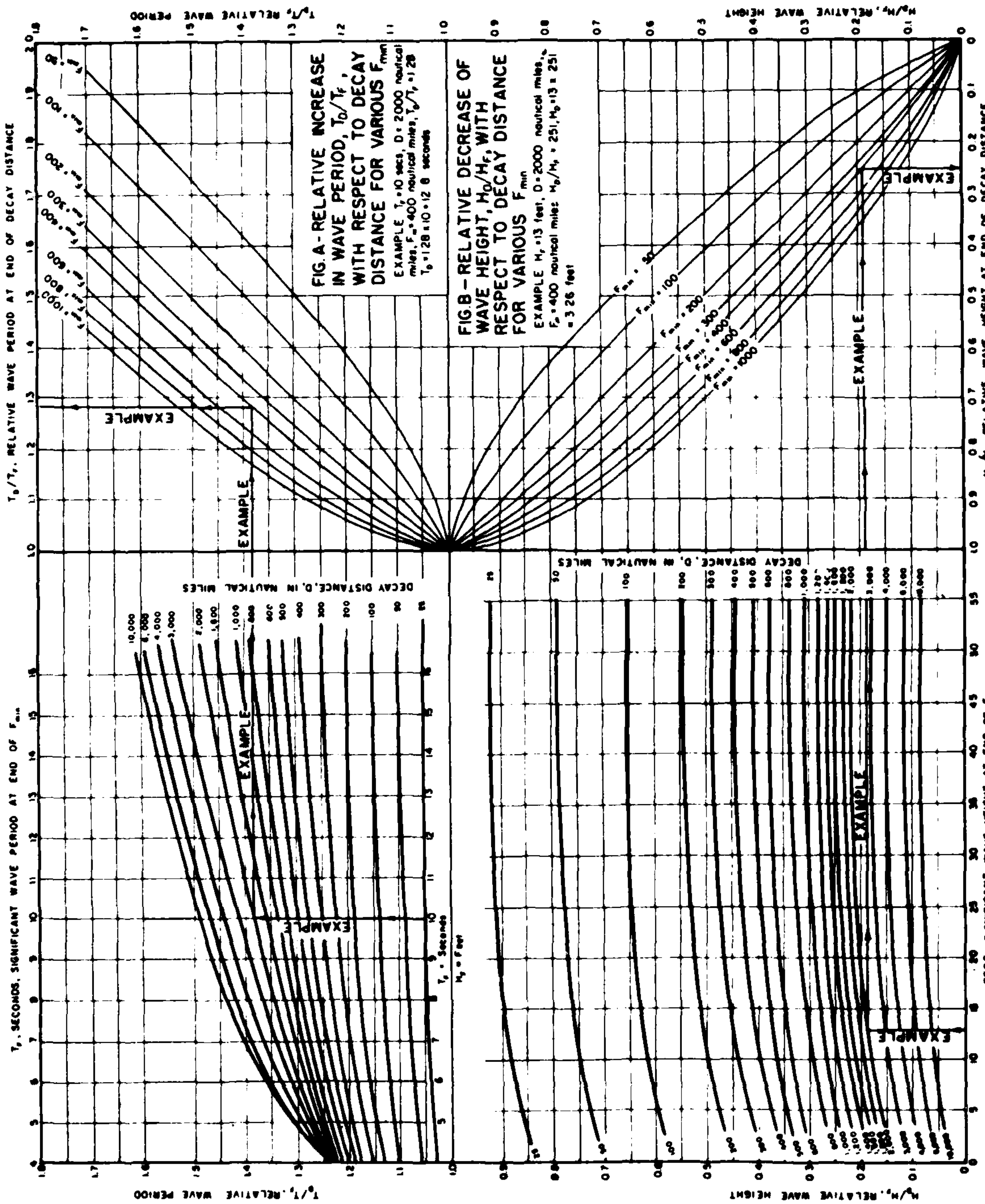


Figure 7-4. Decay curves (from Army CERC, 1973).

When the wave generating area is located a great distance from shore, the wave height and period will change before reaching the nearshore zone. Figure 7-4 presents empirical curves for predicting the increase in period and decrease in wave height corresponding to various decay distances (distance between the leeward side of the generating area and the coast line) and fetch lengths.

7.2.4 Specification of Appropriate Design Wave Height

The wave height (H_s) obtained from statistical analysis of synoptic weather charts and wave forecasting techniques is usually the average of the highest 1/3 of all waves. If the wave heights resulting from the design storm conditions are assumed to conform to a Reyleigh wave height distribution curve, then the height of the design wave can be obtained from the following relationships:

$H_{1/3} = H_s$ = average of highest 1/3 of all waves

$H_{10} = 1.27\ H_s$ = average of highest 10% of all waves

$H_1 = 1.67\ H_s$ = average of highest 1% of all waves

The selection of a design wave height from these relationships depends upon whether the structure is defined as rigid, semirigid, or flexible (Army CERC, 1973). As a rule of thumb, an ocean cable structure is classified as rigid if it is to be immobilized with seafloor fasteners and designed for the maximum allowable load criterion. In this case, excessive hydrodynamic loads may result in failure of either the cable or immobilization system; therefore, the design wave should normally be based on H_1. An ocean cable immobilized with seafloor fasteners and designed for the maximum allowable deflection criterion

may be classified as semirigid, since it can usually absorb some additional loading on an infrequent basis without resulting in failure. For this type of structure the design wave should be based on H_{10}. For flexible systems, such as cables stabilized with mass anchors, the design height is usually the significant height H_s, since mechanical damage due to excessive loading is not likely to occur and the probability of numerous design storms occurring during the life of the installation (resulting in excessive abrasion) is small.

7.2.5 Establishing an Initial Seaward Analysis Point Along The Cable Path

A desirable alternative to beginning the design of a cable stabilization system at an arbitrary water depth or distance from shore is that of determining the approximate depth at which an armored shore end cable begins to become unstable due to hydrodynamic loading. In order to simplify the calculation of the initial analysis point, the effects of shoaling and refraction have been neglected, and, therefore, the water depth obtained is only approximately correct. However, it is sufficiently accurate in most realistic design conditions and eliminates excessive calculations that result from arbitrarily selecting too deep an initial point or neglecting a critical part of the cable by starting the analysis in too shallow a water depth.

The water depth at the initial analysis point can be obtained by solving Equation 6-3b (stability equation) for u_{max}, setting it equal to Equation 5-25 (maximum water particle velocity equation), and solving for $\sinh(2\pi d/L)$, such that:

$$\sinh\left(\frac{2\pi d}{L}\right) \geq \sin\theta\left[\frac{\rho D(C_D + \mu C_L)}{2\mu_s W_s^*}\right]^{1/2} \tag{7-4}$$

where θ is the angle of incidence of the wave orthoginal and the cable path. Assuming that $H = H_o$, a numerical value can be obtained for $\sinh(2\pi d/L)$ for a given cable and design wave parameters. The corresponding value of d/L_o is then obtained from Appendix C, and the desired depth is calculated from

$$d = \frac{g\,T^2}{2\,\pi}\left(\frac{d}{L_o}\right) \tag{7-5}$$

7.2.6 Establishing Water Depth and Wave Length

For the initial analysis point, the water depth is defined by Equations 7-4 and 7-5. The actual wave length at this depth is calculated from Equation 5-29 as discussed in Section 5.4.3. As the analysis proceeds shoreward, a new analysis point must be selected by either specifying a new shallower depth or by moving a fixed distance shoreward and determining the depth from the topographic model discussed in Section 7.2.1. Care must be taken when implementing either of these procedures not to overlook critical points that may occur on irregular seafloors.

7.3 SHOALING AND REFRACTION ANALYSIS

The actual wave height at the analysis point is next determined by taking into account the effects of shoaling and refraction. If the seafloor contours developed for the topographic model are essentially straight and parallel, the combined effects of shoaling and refraction may be determined analytically as described in Section 5.4.4. If on the other hand the contours do not match this ideal condition, the effects

of refraction must be determined graphically from the orthoginal construction techniques (Section 5.4.5), and the shoaling effects determined separately from the procedure outlined in Section 5.4.3.

7.4 BREAKING WAVE ANALYSIS

Since none of the wave theories are valid for waves once they have broken, the height of the wave calculated in Section 7.3 must be compared to the maximum wave height that could exist at that point without actually breaking (Section 5.4.6). If the wave height is less than the breaking wave height (H_b) determined from Figure 5-9, the analysis is carried out using the wave height calculated in Section 7.3. If on the other hand the breaking wave height is exceeded, then an imaginary wave is developed whose height is equal to the maximum nonbreaking wave that could exist at that point (from Figure 5-10). The hydrodynamic force analysis is then carried out using this imaginary wave height and a corrected drag force coefficient (multiplied by 2.5). Although there is no known theoretical basis for this procedure, the results obtained by this method appear to be reasonable when compared to experimental results (see Section 5.4.1).

7.5 HYDRODYNAMIC FORCE ANALYSIS

7.5.1 Water Particle Motion Calculations

The maximum water particle velocity and acceleration at the seafloor are calculated by substituting the wave parameters obtained in the previous section into Equations 5-25 and 5-26. A decision as to the relative importance of the inertia force can now be made by evaluating the ratio:

$$\frac{F_I}{F_H} = \frac{3 \pi D}{2 H} \sinh \left(\frac{2 \pi d}{L}\right) \quad (7\text{-}6)$$

For values of F_I/F_H less than 0.1, the calculation of $\dot{u}_{max}$ and F_I may, for all practical purposes, be deleted from the hydrodynamic force calculations.

7.5.2 Resultant Velocity Vector

The maximum velocity magnitude and direction resulting from the combined effects of both wave- and current-induced water particle motion are calculated from Equations 5-37 and 5-38 (see Section 5.4.7).

7.5.3 Selection of a Stabilization Technique

A stabilization technique must be selected from the list of feasible mass anchor systems obtained as a result of the site evaluation analysis conducted in Chapter 3. At this stage of the analysis, the important parameters associated with the mass anchor systems are the vertical cross section area per unit length (equal to the diameter for cylindrical objects), the submerged weight per unit length, and the force coefficients. The area and submerged weight data are found in the appropriate part of Section 4.2 (mass anchors), and the base force coefficients for cylindrical bodies are tabulated in Table 5-2. Force coefficients for split-pipe must be determined from the procedure discussed in Section 5.5.9.

7.5.4 Determine Appropriate Force Coefficients

The actual force coefficients to be used for the hydrodynamic analysis must now be obtained by evaluating the effects of angle of incidence and clearance between the mass anchor system and the seafloor. Figures 5-17 and 5-18 present reduction coefficient values for

various angles of incidence and clearances above the seafloor, respectively. The combined force coefficients are calculated by substituting these correction coefficients and the base hydrodynamic force coefficient values into Equations 5-42 and 5-43.

7.5.5 Hydrodynamic Force Calculations

The net horizontal hydrodynamic force acting on the cable due to the combined effects of wave- and current-induced water particle motion are obtained by substituting the system parameters, coefficient of friction, combined force coefficients, and maximum water particle velocity into Equation 6-3b. If the inertia force was found to be a significant factor in the total hydrodynamic loading (Equation 7-6), then the water particle acceleration must also be included, and the net horizontal force calculated from Equation 6-3a.

7.6 STABILITY ANALYSIS

The stability of a cable/mass anchor system is determined by applying the criteria described in Section 6.2. For negative values of F_H^*, the maximum hydrodynamic force will not exceed the maximum friction force between the mass anchor system and the seafloor; the cable will remain stable at the point of analysis under the influence of the design wave and current loads. For positive values of F_H^*, the system will be unstable and will move along the seafloor under the influence of the hydrodynamic forces. For this condition, the design engineer has two options: (1) select a new mass anchor system and repeat the analysis for the same point, or (2) select a feasible immobilization technique and proceed with an immobilization system design. If the first option is selected, the analysis must be repeated beginning with Section 7.5.3. The second option, immobilization system design, is discussed in the following section.

7.7 IMMOBILIZATION SYSTEM DESIGN

A feasible immobilization technique is selected from the list compiled from the site evaluation procedure discussed in Chapter 3. If a mass anchor system is to be used in conjunction with an immobilization technique, then the effect of hydrodynamic loads on the mass anchor system must be analyzed prior to beginning the immobilization system design.

The maximum design loading condition is established by evaluating the loading factor parameter χ (Equation 6-25). For values of $\chi > 1$, Load Condition 1 (Equation 6-23) is utilized to develop the appropriate design equations, while for values of $\chi < 1$, Load Condition 2 (Equation 6-24) is used.

The design criteria (either maximum allowable load or maximum allowable deflection) must be established. Since both of these criteria must be satisfied simultaneously to assure an uninterrupted operation during the life of the system, the analysis will normally require that a design be calculated for both criteria and the more conservative of the two designs be utilized for the installations. The specific equations to be used for the design of the immobilization system are developed by evaluating the initial cable tension condition (T_I) and substituting this value into the general equation resulting from the selection of the design criteria and the evaluation of the loading factor parameter (see Section 6.5).

Once the protection system has been designed to either stabilize and/or immobilize the cable for the conditions existing at the analysis point, a new position along the cable path is selected, and the analysis procedure is repeated beginning with Section 7.2.6. This stepwise analysis procedure is repeated for all of the preselected points, moving shoreward along the cable path until either the shoreline is reached or the cable enters protected water (i.e., a harbor or lagoon).

7.8 ECONOMIC ANALYSIS

The final phase of the nearshore cable protection design is an economic analysis of the various feasible alternatives. To reach an optimum solution, from an economic point of view, the actual protection system may require a combination of stabilization, immobilization, and burial techniques.

Due to the numerous variations of installation techniques available for each protection system, factors imposed by specific site conditions, changes in technology which constantly make new tools and techniques available, and the effect of inflation, a detailed procedure for conducting this economic analysis is beyond the scope of this manual. However, an outline of factors which, in the past, have influenced the cost of implementing some of the various cable protection techniques is presented below as a guide to conducting this phase of the analysis.

I. Survey of Potential Sites

- A. Define mission requirement and constraints of the cable system being installed.
- B. Develop site survey plan.
- C. Locate and assemble personnel and equipment required to conduct site survey.
- D. Perform survey of one or more candidate sites.
    1. Contact local sources for first-hand information.
    2. Perform literature search for environmental conditions.
    3. Perform underwater survey with divers.
    4. Determine logistical requirements:
        - a. Surface support craft and recompression facilities
        - b. Messing and berthing facilities
        - c. Transportation and per diem
        - d. Site survey report

II. Engineering Analysis

A. Analyze site survey data.

B. Select potential cable route and feasible cable protection techniques.

C. Prepare preliminary design of two or more protection techniques for cost comparison.

D. Select an optimum technique.

E. Prepare a detailed design of the protection system and plan the installation procedure (may require gathering of additional environmental data).

III. Project Mobilization

A. Procure materials.

1. Expendable
2. Reusable

B. Contract for various project phases or elements (i.e., trenching on land, surface ship support).

C. Determine location and cost of equipment to be borrowed or rented.

D. Locate and arrange for specialized labor (on-site representatives, specialized contracting services).

E. Modify existing equipment for specialized site requirements and prepare for transit.

F. Repair or purchase missing parts. Accumulate necessary spare parts required for duration of deployment.

IV. Transit

A. Truck to ship or air-shipment point.

B. Ship by air or sea (Note: During shipment, contractors' equipment usually bear reduced rental rate).

C. Truck or sea-transit to on-site storage area.

D. Temporarily store in staging area until all equipment and personnel arrive.

E. Supply transportation and per diem for construction personnel.

V. Setup

A. Construct access roads to work site and prepare beach and additional work site areas.

B. Construct or move in trailers for office, work space, and diving equipment area.

C. Construct foundations, bases, etc., for equipment.

1. Beach anchors for winches
2. Beach anchors for snatch blocks
3. Foundations for large equipment
4. Cable guides and beach mats where installation and immobilization require work on beach
5. Junction box/vault construction

D. Install reference markers for the cable route.

1. Range markers on beach
2. Buoy in the water

E. Install mooring for work boats.

1. Cable ship or barge
2. Diving support boats

F. Assemble specialized hardware and equipment (i.e., drilling rig, trenching machines, etc.)

VI. Immobilization System Installation

Because of the numerous techniques available for installing and protecting cables and the modification of these techniques to meet specific site requirements, the reader is referred to the appropriate sections of Chapter 4 to determine the various elements involved in implementing

a specific technique. Guidelines on anticipated production rates and personnel requirements for each of these protection techniques are also presented in Chapter 4.

VII. Cleanup

A. Restore beach and work site to the preinstallation condition.

B. Disassemble and pack equipment.

C. Inspect installation and document as-built configuration.

D. Return equipment by truck/air or ship to point of origin.

E. Transport personnel.

VIII. Post-Deployment Operations

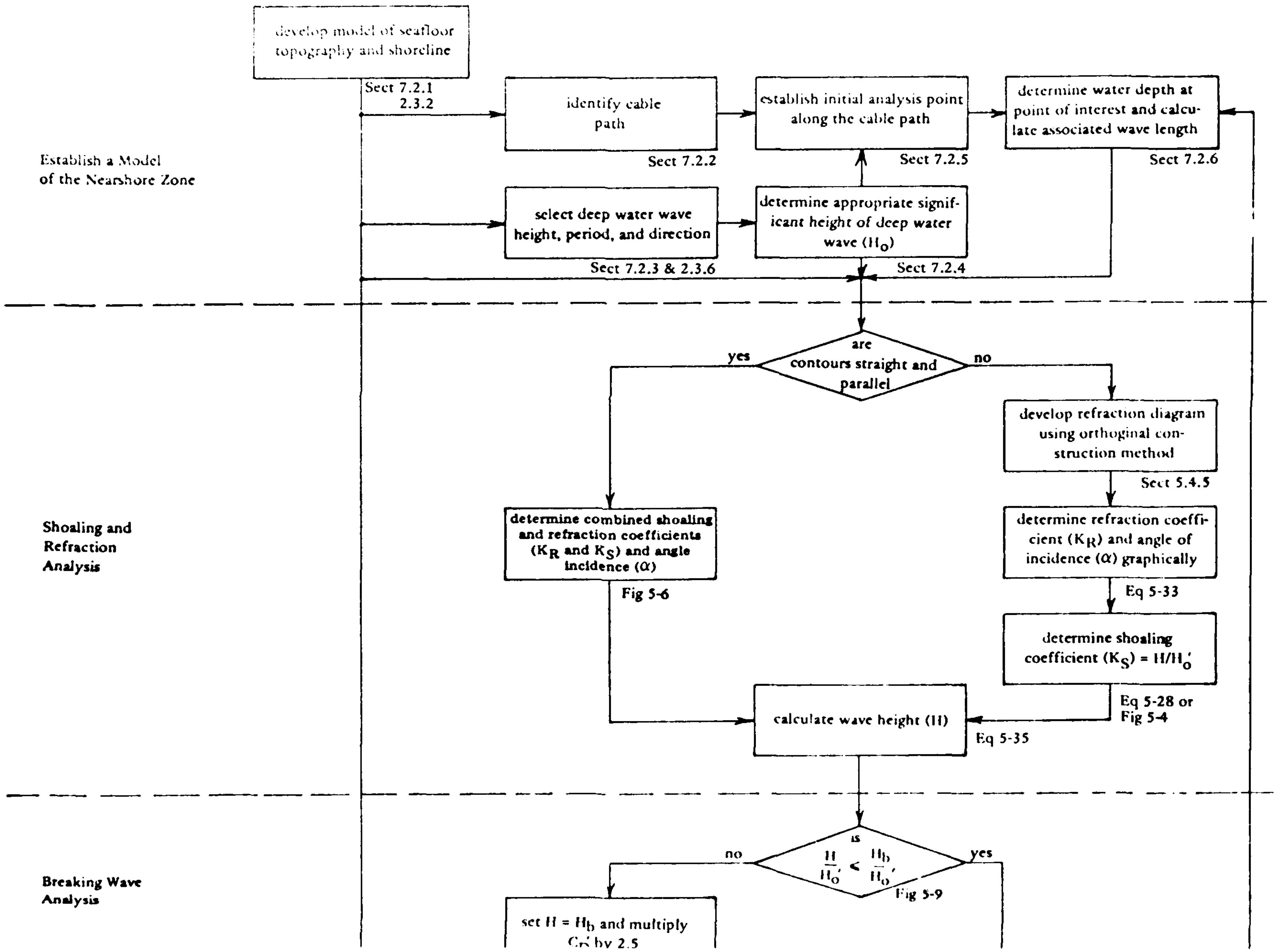
A. Repair, maintain, or replace equipment utilized on project.

B. Document operation.

7.9 SUMMARY

This chapter has presented a step-by-step procedure for designing and conducting an economic analysis of a nearshore cable protection system. This approach is only one of many that could be used to analyze the effects of hydrodynamic forces on bottom-resting cables and the subsequent design of stabilization/immobilization systems which will allow the cable to resist these forces. This procedure was selected because it can be easily adapted to both hand calculation or computer solution of the system design problem.

A flow chart that summarizes the steps in this design process is presented in Figure 7-5. The pertinent sections of the design guide and equations required for computations in each step have been referenced as an aid to carrying out the analysis procedure. In order to facilitate computations the figures, graphs and tables used in the design of cable protection systems have been compiled in Appendix D, and all of the equations presented in the text have been reproduced in Appendix E.



Establish a Model of the Nearshore Zone
develop model of seafloor topography and shoreline
Sect 7.2.1 2.3.2
identify cable path
Sect 7.2.2
establish initial analysis point along the cable path
Sect 7.2.5
determine water depth at point of interest and calculate associated wave length
Sect 7.2.6
select deep water wave height, period, and direction
Sect 7.2.3 & 2.3.6
determine appropriate significant height of deep water wave (H_o)
Sect 7.2.4
Shoaling and Refraction Analysis
are contours straight and parallel
yes
no
develop refraction diagram using orthoginal construction method
Sect 5.4.5
determine refraction coefficient (K_R) and angle of incidence (α) graphically
Eq 5-33
determine shoaling coefficient (K_S) = H/H_o'
Eq 5-28 or Fig 5-4
determine combined shoaling and refraction coefficients (K_R and K_S) and angle incidence (α)
Fig 5-6
calculate wave height (H)
Eq 5-35
Breaking Wave Analysis
is $\frac{H}{H_o'} < \frac{H_b}{H_o'}$
Fig 5-9
no
yes
set $H = H_b$ and multiply

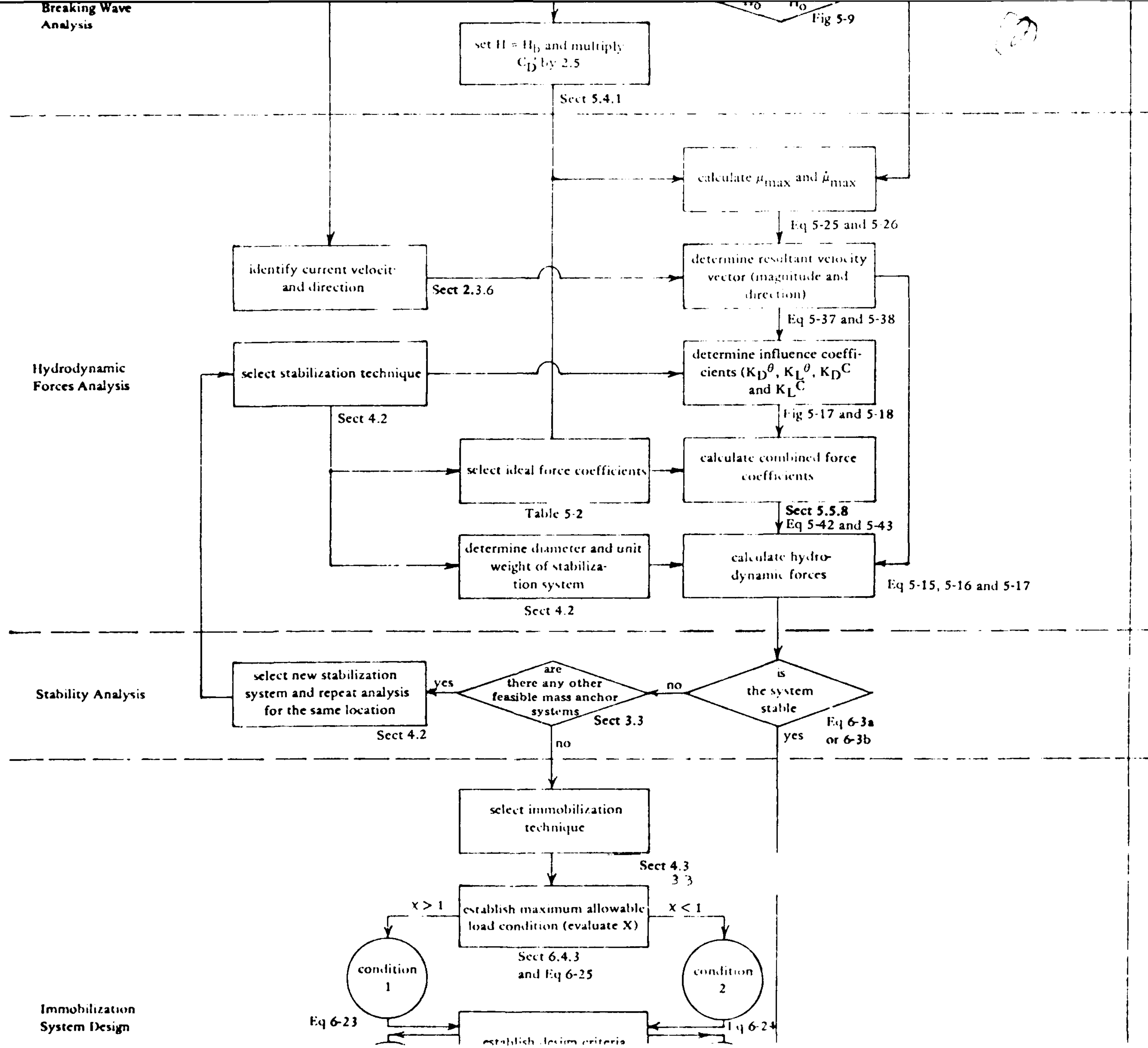
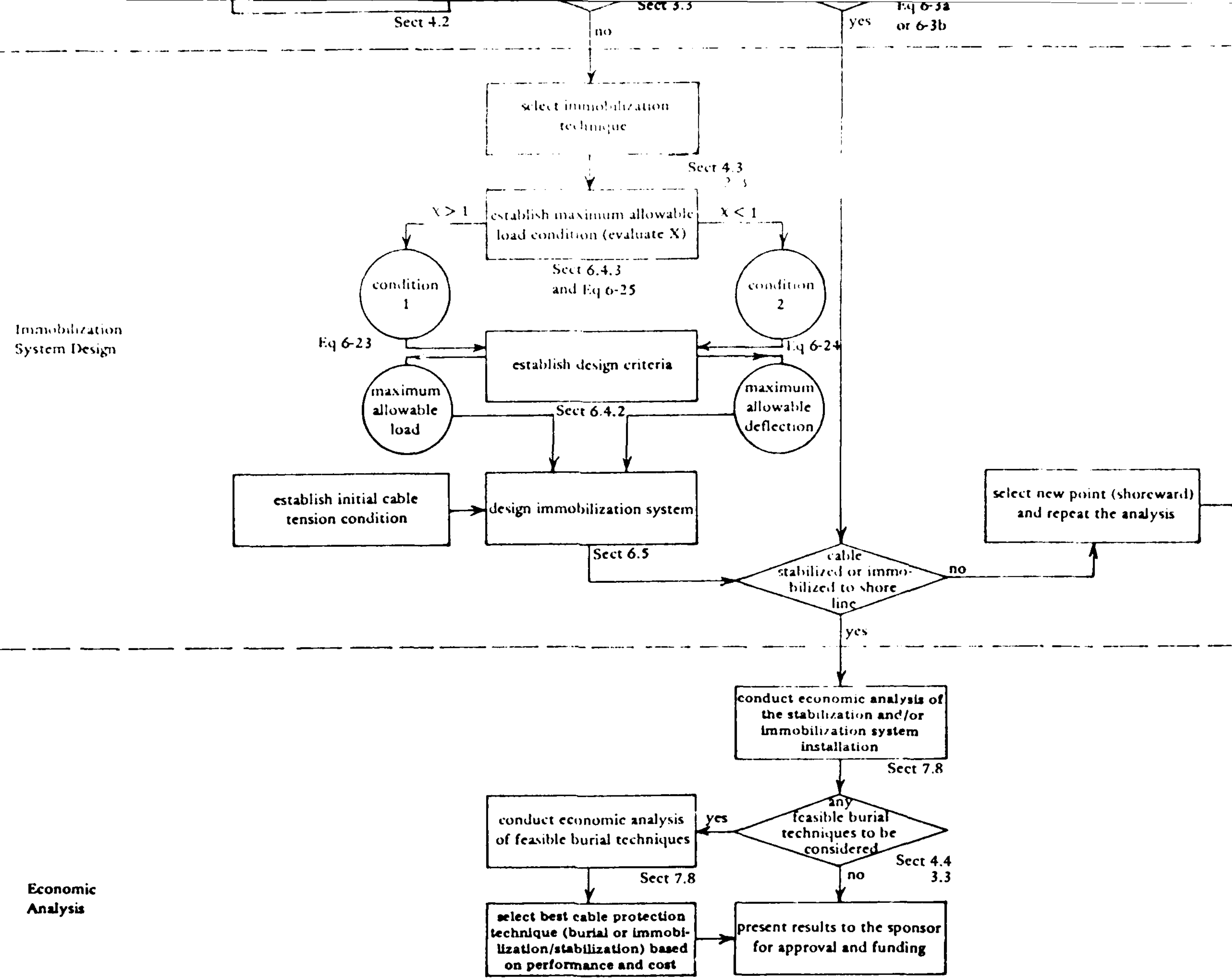


Figure 7-5. Cable protection system analysis and design procedure.


Sect 4.2
Sect 3.3
no
yes
Eq 6-3a
or 6-3b
Immobilization
System Design
select immobilization
technique
Sect 4.3
establish maximum allowable
load condition (evaluate X)
X > 1
X < 1
Sect 6.4.3
and Eq 6-25
condition
1
condition
2
Eq 6-23
Eq 6-24
establish design criteria
maximum
allowable
load
maximum
allowable
deflection
Sect 6.4.2
establish initial cable
tension condition
design immobilization system
Sect 6.5
cable
stabilized or immo-
bilized to shore
line
no
yes
select new point (shoreward)
and repeat the analysis
Economic
Analysis
conduct economic analysis of
the stabilization and/or
immobilization system
installation
Sect 7.8
any
feasible burial
techniques to be
considered
yes
no
Sect 4.4
3.3
conduct economic analysis
of feasible burial techniques
Sect 7.8
select best cable protection
technique (burial or immobi-
lization/stabilization) based
on performance and cost
present results to the sponsor
for approval and funding

Appendix A

PROCEDURES FOR WELDING NODULAR CAST IRON SPLIT-PIPE
(Adapted from Welding Handbook 5th Edition - Section 4)

Cast iron is an iron-based material that also contains carbon, silicon, manganese, phosphorus and sulphur. The carbon is present in two forms: as combined carbon (as in steels) and as free carbon (graphite). In nodular cast iron the free carbon has a spheroidal appearance.

When cast iron is heated, as during welding, the iron may absorb some of the free carbon; upon rapid cooling, the heat-affected zone can become very hard and brittle. Because of this metallurgical factor, cast iron is more difficult to weld than carbon steel.

SHIELDED METAL ARC WELDING

Joint Preparation

The joint between the two sections of pipe to be welded should be machined or ground to produce a 60-degree included angle. The root opening should permit fusion with the root faces and must be sufficiently wide for uninterrupted electrode manipulation. Castings found to contain large amounts of gas should be baked for a few minutes at 1000°F or heated with acetylene or other oxy-fuel torches prior to welding.

Preheat and Welding

Nickel-base welding electrodes are widely used for the welding of gray, nodular and malleable cast irons. The weld metal of both the ENi and ENiFe type electrodes have a carbon content above the solubility limit. As the weld metal solidifies, the excess carbon is rejected as graphite causing an increase in volume; this lessens shrinkage stresses. Generally, the ENiFe electrode is considered superior to the ENi type because:

1. The coefficient of expansion in the weld metal is decreased.
2. Tolerance for phosphorus in the base metal is increased.

3. The strength and ductility of the weld deposit is increased.

4. There is very little tendency, if any, for hot-cracking.

Welds made using the ENiFe electrode should be preheated to 300° to 400°F. Weld deposits appear to be stronger and more ductile when made with large diameter electrodes. The welding current should be the same or slightly higher than the electrode manufacturer's recommendation (about 185 amps, DCRP* for a 3/16-in.-diam electrode).

Stress Relief

A welded nodular iron casting should be anealed. Immediately after welding, the part can be transferred to a hot furnace between 400° and 800°F and heated to 1650°F; it should be held at that temperature for 2 hours and then slowly cooled to 1300°F, held there for 5 hours, and then cooled to room temperature. The part may also be removed and cooled in still air after having reached 500°F. In furnace heating and cooling, the temperature change of the part should not exceed 150°F per hour.

OXYACETYLENE WELDING

It is more difficult to weld nodular iron than gray iron by the oxyacetylene welding process because of the formation of gas pockets. This difficulty is associated with the boiling (vaporization) of the residual magnesium. Its boiling point is 2025°F, which is lower than the melting point of the nodular iron. Cerium has a boiling point above 6000°F and is used in the filler metal to nodularize the weld deposit.

Joint Preparation

Grooving or removing defective metal should be done by chipping or machining or through the use of a rotary file. Grinding wheels or gas cutting should not be used. A 90-degree included groove angle is satisfactory if a double-vee groove is used. Where the weld can be made from only one side, the included groove angle may be increased to 120 degrees. There must be sufficient width to permit root face fusion and uninterrupted torch manipulation.

Filler Metals, Preheating and Welding

A preheat of 1100° to 1200°F should be maintained. Furnace preheating is desirable, and it can be maintained by using auxiliary low-intensity burners during welding. It is also possible to mount the part

*Direct current, reverse polarity.

over low-intensity burners and cover the casting with asbestos or other fire-resisting materials. If the preheat falls below 1100° to 1200°F, the welding torch will have to supply more heat to fuse the weld deposit. This increases the likelihood of producing gas pockets when the base metal is melted. It is imperative to melt as little base metal as possible and maintain a "cold puddle." The low-intensity auxiliary burners should be able to maintain the preheat so that heat input with the welding torch can be kept low. The use of cerium-bearing, rather than magnesium-bearing, nodular filler is recommended.

Maintaining a "cold puddle" is the key to the gas welding technique. The flame should be directed primarily over the tip of the welding rod instead of on the puddle. The wetting and fusing with the base metal is achieved by scraping or rubbing with the filler rod. As the layers are deposited, care must be exercised to melt as little of the base metal as possible. Flux is added as required.

Stress Relief

Immediately after welding, the welded casting should be placed into a 1100° to 1200°F furnace and be heated to 1650°F, held at this temperature for 2 hours; then it should be furnace cooled to 1300°F, held there for 5 hours and then cooled to room temperaure. The part may also be air cooled after furnace cooling to 500°F.

Appendix B

SUGGESTED MATERIALS AND EQUIPMENT FOR SPLIT-PIPE APPLICATION

UNDERWATER INSTALLATION

Description	Quantity
Appropriate diving equipment	as required
Jumper cables	
5-gal gas can w/spout	3
Extra K-bars	12
First aid kit	2
2000-lb lift bag	1
Float balloons (for lift bags), misc	10
Diver tool bag	10
Photo gear	as required
Towels	as required
Sledge hammer	
LARC compass	1
Silicone spray	as required
Pry bar	2
Small buoys	25
WD-40	24 cans

Description	Quantity
Garden hose	as required
PRC-77's (or 25's) w/antenna	6
Hand sets	6
Headsets	6
Extra batteries	36
Extra antenna	2
PRC-77 amplifier units (for jeep or LARC)	4
110V power supply for PRC-77	1
Float balloon fillers	2
K-bottle charging lead	1
Hydraulic power source	1
Hydraulic oil (MIL-H-24430 or MIL-H-5606C)	55 gal
Hydraulic hose on reel	200 ft
Hydraulic hose, 50-ft lengths	2 ea
Hydraulic fittings, misc	
Spare parts for hydraulic power source	as required
Hydraulic impact wrenches	2
Hydraulic grinder	1
Hand pump (oil drum)	1
Disc for hydraulic grinder	12
Complete mechanics tool box w/lock	1
Hydrometer	1
VOM	1

Description	Quantity
Flashlight	2
Scuba bottle tire filling attachment	2
Grease gun, full	1
Valve stem core remover	4
Black tape	6
Assorted nuts, bolts, screws	as required
3/4-in. electric drill w/bits	1
Zodiac	2
Oars	4
Foot pump	2
Zodiac patch kit	2
Zodiac pressure gauge	2
5-lb anchor	2
Zodiac scuba bottle filling attachment	
25-hp outboard	2
9-hp outboard	1
3-gal gas tanks	4
Outboard motor oil	4 cases
Outboard spare parts	as required
1/4-in. shackles	10
3/8-in. shackles	30
1/2-in. shackles	25
3/4-in. shackles	12

Description	Quantity
1-in. shackles	6
Seizing wire	25 ft
3/4-in. wire rope	as required
3/8-in. wire rope	as required
1-in. diam line/wire rope snatch block	3
6 thread	1 coil
15 thread	2 coil
21 thread	3 coil
3-in. nylon	3000 ft
1-1/2-in. nylon	1 coil (600 ft)
1-1/2-in. polypro	1 coil (600 ft)
Air tugger winches (1200 lb)	2
3/4-in. polypro	1 coil (600 ft)
Marlin spikes	
3/4-in. wire rope clips	100
3/8-in. wire rope clips	120
3/4-in. wire rope thimbles	30
3/8-in. wire rope thimbles	23
15 thread	1 coil
Work gloves, rubber-coated	20 pr
Wire rope cutter	1
Turnbuckles, 1/2 in.	20
Pelican hooks (10 ton)	2

Description	Quantity
Project files	as required
Math tables	1 ea
Calculator w/charger	1 ea
Electric heaters	4
Padlocks	6
Bull horn w/batteries	1
Extension cord	as required
Crescent wrenches and assorted tools	as required
Boat hooks	5
Fathometer	1
Binoculars	2 pr
15/16 socket, deep/thin wall	24
Ratchets, spud	10
Open end 15/16 spud wrenches	12
Small BTL stopper	1
Recompression chamber	1
Transit w/accessories	1
Come-along (1-1/2 ton)	2
Grip hoist, T20, 3300-lb capacity, FSN 3950-729-6165	1
Grip hoist, T15, 1650-lb capacity	1
Portable tool room	1
Pneumatic impact wrench	2
Floating flashers	4

Description	Quantity
Ladder, LARC V diving	1
ARCAIR electric cutting torch, model H-Z Cat. #61-042-000, Arcair Co., Bremerton, CA 98313	
Arcair 1/4-in. rods	2 boxes
Sextant, nautical	1 ea
Probe, underwater signal, WECO	1 ea
Anchor, mushroom, 500-lb	2 ea
Stopper, chain, 3/4-in. tapered	3 ea
Buoy, crown, steel, 150-lb	8 ea
Balloon, flotation, cable per Pashings DWG 520-D-1543479	as required
Propellers, LCM-6	2 ea
Shaft, propeller, LCM-6	1 ea
Compass, No. 5, Mark 2 Mod 0, FSN 46605-255-0238 (for LCM-6)	1 ea
Tempstick, assorted ranging, 300°F to 1250°F	18 ea
Rod, welding, Nickel-ARC, 55-AC-DC 5/32 in., L204B27DE 4E, 5-lb can	13 can
Rod, welding, Nickel-ARC 55-AC-DC 3/16 in., AZ04B27D ZE, 5-lb can, CHEMTRON CORP., Hanover, PA	17 can
Table, 4x6-ft top	2 ea
Chairs, folding	8 ea
Generator, diesel, 120 VAC 60 cycle, 30 kW	1 ea
Van, personnel carrier 0090 with electric power, lights and telephone (Beach Station, if required)	1 ea

Description	Quantity
Space heaters	as required
Truck, 6-passenger, pickup	1 ea
LCM-6	1 ea
Fork lift, rough terrain, occasional use	1 ea
Light plant, with lights	1 ea
Tarpaulins, 14x14 ft	3 ea
Purchase, two-fold, 2000-lb	2 ea
Chain, 5/8-in.	8 ea
Diesel fuel tank, 500-gal	1 ea
Concrete clump, 3x3x3-ft reinforced, with 1-in. lifting padeye	as required
Anchor, 4-fluked, "Rock Hook," 50-lb	2 ea
Chalk board	1 ea
Litter, stokes	1 ea
Torque wrench, 150 ft-lb	2 ea
Pump, portable, gasoline-operated, 500-gpm	1 ea
Grease, boat trailer and marine, stalube GB-41, Compton, CA, 1-lb can	10 ea

SHORE INSTALLATION

Description	Quantity
Mushroom anchor, 450-lb	2 ea
Johnson Messenger Transceivers or equivalent with spare batteries and headsets	4 ea
Sheaves, aluminum, 36-in.-diam w/Axel Padeyes	3 ea

Description	Quantity
Tapered Chain Stoppers (certified), 3/4-in.-diam chain	2 ea
1-in. bull chains, approx 8 ft long	3 ea
Thimbles, galvanized, for 3/4-in. wire rope, extra heavy pattern, Crosby-Laughlin No. G-414 or equivalent	16 ea
3/4-in. wire rope clips	30 ea
3/4-in. 6x37 IWRC improved plow steel wire rope	600 ft
Floats (Navy Standard Stock No. H2050-574-5963)	20 ea
Manila, 21 thread	1 coil
BTL stoppers	2 ea
Protector, half-section, armor submarine cable (split-pipe)	634 ea
Bolt, machine, sq hd 5/8-in. - 11 NC-2Ax2-1/2-in., 15/16 across flats, 18-8 CRES	2600 ea
Nut, HEX, 5/8-in. - 11 NC-2B, 15/16 in. across flats, 18-8 CRES	2750 ea
Washer, lock, spring, 18-8 CRES, medium No. 5/8 in.	2750 ea
Tape, impregnated cotton, KS 16255, list 1, 2-in. wide	2 roll
Rope, polypropylene, 1/4-in.-diam, 3 strand laid construction	1000 ft
Line, nylon no. 6 thread crowline, 38-40 lb/coil, 45-50 ft/lb (Pot Warp.)	6 coil
Screweye, 5/16 in. stock LOA, 4-in. ID Eye, 5/8 in. zinc-plated steel	30 ea

Description	Quantity
Shackle, screw pin, galv., 5/16 in. Crosby-Laughlin Type G-209	30 ea
Sockets, hex, impactool, 5/8 in. sq. drive, thin wall - 7630H, 15/16 in. across flats, for use with 908 impactool	6 ea
Bag, tool collapsible canvas, nonmetallic, 9x14 in. - McMaster Corp., #6596A1	6 ea
Wrench, structural 15/16 in. nominal opening with tapered end, McMaster Carr, #5406A22	6 ea
Wrench, ratchet, 1/2 in. sq. drive, 15 in. long, McMaster Carr, #5523024	6 ea
Adapter, impactool, sq. dr., 5/8 in. male to 1/2 in. female, McMaster Carr, #5549A46	6 ea
Banding machine and banding material for securing BTL stoppers to cable	1 ea
8x3 grapnel rope	3000 ft
Probe for locating cable	1 ea
Portable generator, 10 KVA (or greater) 120-V, 60 Hz on two separately fuzed 30-A circuits	1 ea
Portable air compressor with a 5-ft^3 storage tank, to provide a continuous supply of 20 cfm, 80-psig (min) air. The air shall be filtered and must be oil and dust free with a moisture level of <1%.	1 ea
Compressed air manifold with inlet to fit 1-in. ID air hose and having three Hansen series 1-HK stainless socket female connections No. LL1-H11, 1/8-in. FPT mounted on petcock cut-offs	1 ea
Air hose, 1-in. ID, 50 ft long, one end to fit the compressor, the other to fit the manifold	1 ea

Description	Quantity
TD-20s, at least one must be mounted with a 15-ton winch with 250 ft of 1-1/8 in. IWRC improved plow steel wire rope (new)	2 ea
Jeep	1 ea
LARC V	1 ea
5-ton truck (with sand tires)	1 ea
Back hoe	1 ea
Front-end loader with fork attached	1 ea
Zodiacs (with outboard motors)	2 ea
Spare outboard motor	1 ea
Light Plant	1 ea
Floats (Navy Standard Stock No. H2050-574-5963)	120 ea
2000-ft distance line	1 ea
Bolts, machine, sq. head, 5/8 in. - 11 NC-2Ax5 in., 15/16 across flats, A325 steel	10 ea
Bolts, 5x7/8 in. - 9 NC A325	6 ea
Bolts, 10x3/4 in. - 10 NC	10 ea
3/8-in. screw pin anchor shackles	12 ea
3/4-in. screw pin anchor shackles	6 ea
1-in. screw pin anchor shackles	13 ea
1-1/4-in. screw pin safety anchor shackles	4 ea
3/4-in. wire rope clips (in addition to 30 provided by WECO)	120 ea
Galvanized thimbles for 3/4-in. wire rope (in addition to 16 provided by WECO)	20 ea

Description	Quantity
3/4-in. IWRC improved plow steel wire rope (in addition to the 600 ft provided by WECO)	900 ft
1-1/8-in. improved plow steel wire rope, must have eyes in both ends	500 ft
21 thread line	4000 ft
Impact wrenches (pneumatic), 5/8 or 1/2 in. sq. drive	6 ea
Air compressor to operate the 6 impact wrenches; manifold and hose should also be provided	1 ea
Fiberglass matting (MO Matting), thirteen 40-ft sections, 9 ft wide	520 ft
Ready Mix Concrete, 5000 psi, procured locally	3 yd^3
Lumber and fasteners necessary to construct range markers, forms for beach anchor, cable hauling guides, beach deadmen, and ramp for road crossing; may be procured locally	L.S.
Grapnel rope	1 ea
Ship anchorage marker buoy	1 ea
Jet pump, hose, and nozzle	1 ea
3/4-in. mild steel brackets	8 ea
55-gal drums (empty); may be procured locally	2 ea
Transceivers	8 ea
Fire hose, preferably discarded, size to be determined later	500 ft
Barbed wire (to repair fence taken down when hauling in the cable)	1 roll

Description	Quantity
Grass seed; Perennial Rye, 45%; Subterranean Clover, 45%; Alsike Clover, 10%	40 lb
Cable cutters, for cutting 3/4-in. wire rope	3 ea
Sledge hammers, 10-16	2 ea
Cable sled to lift cable to allow placing split pipe; must not cause less than 10-lb bend radius in the cable	1 ea
Cable reel jacks (for hauling grapnel line)	2 ea
Cable reel spindle	1 ea
3/4-in. manila line	100 ft
Pelican Hook, 10-ton	1 ea
Seizing wire	50 ft (1 roll)
Transit	1 ea
Level	1 ea
Range Poles	2 ea
1-1/8-in. wire rope clips	24 ea
1-1/8-in. wire rope thimbles	6 ea
Prybars, 5 ft	4 ea
Concrete clump anchors, 40-lb	30 ea
Lengths of lumber, 7 ft x 3/4 in. x 3/4 in.	30 ea
1-ft lengths of 1/2-in. chain	30 ea
Riggers forming vise	1 ea
Spring tensiometer (50,000-lb capacity)	2 ea

Description	Quantity
3/8-in. wire rope	200 ft
3/8-in. wire rope thimbles	12 ea
3/8-in. wire rope clips	48 ea
1/2-in. anchor chain	90 ft
Appropriate diving equipment	as required

Appendix C

FUNCTIONS OF d/L FOR EVEN INCREMENTS OF d/L_o
(After Wiegel, 1964)

d/L_o	d/L	$2\pi d/L$	TANH $2\pi d/L$	SINH $2\pi d/L$	COSH $2\pi d/L$	H/H'_o	d/L_o	d/L	$2\pi d/L$	TANH $2\pi d/L$	SINH $2\pi d/L$	COSH $2\pi d/L$	H/H'_o
0	0	0	0	0	1	∞	.006000	.03110	.1954	.1929	.1967	1.0192	1.620
.0001000	.003990	.02507	.02506	.02507	1.0003	4.467	.006100	.03136	.1970	.1945	.1983	1.0195	1.614
.0002000	.005643	.03546	.03544	.03547	1.0006	3.757	.006200	.03162	.1987	.1961	.2000	1.0198	1.607
.0003000	.006912	.04343	.04340	.04344	1.0009	3.395	.006300	.03188	.2003	.1976	.2016	1.0201	1.601
.0004000	.007982	.05015	.05011	.05018	1.0013	3.160	.006400	.03213	.2019	.1992	.2033	1.0205	1.595
.0005000	.008925	.05608	.05602	.05611	1.0016	2.989	.006500	.03238	.2035	.2007	.2049	1.0208	1.589
.0006000	.009778	.06144	.06136	.06148	1.0019	2.856	.006600	.03264	.2051	.2022	.2065	1.0211	1.583
.0007000	.01056	.06637	.06627	.06642	1.0022	2.749	.006700	.03289	.2066	.2037	.2081	1.0214	1.578
.0008000	.01129	.07096	.07084	.07102	1.0025	2.659	.006800	.03313	.2082	.2052	.2097	1.0217	1.572
.0009000	.01198	.07527	.07513	.07534	1.0028	2.582	.006900	.03338	.2097	.2067	.2113	1.0221	1.567
.001000	.01263	.07935	.07918	.07943	1.0032	2.515	.007000	.03362	.2113	.2082	.2128	1.0224	1.561
.001100	.01325	.08323	.08304	.08333	1.0035	2.456	.007100	.03387	.2128	.2096	.2144	1.0227	1.556
.001200	.01384	.08694	.08672	.08705	1.0038	2.404	.007200	.03411	.2143	.2111	.2160	1.0231	1.551
.001300	.01440	.09050	.09026	.09063	1.0041	2.357	.007300	.03435	.2158	.2125	.2175	1.0234	1.546
.001400	.01495	.09393	.09365	.09407	1.0044	2.314	.007400	.03459	.2173	.2139	.2190	1.0237	1.541
.001500	.01548	.09723	.09693	.09739	1.0047	2.275	.007500	.03482	.2188	.2154	.2205	1.0240	1.536
.001600	.01598	.1004	.1001	.1006	1.0051	2.239	.007600	.03506	.2203	.2168	.2221	1.0244	1.531
.001700	.01648	.1035	.1032	.1037	1.0054	2.205	.007700	.03529	.2218	.2182	.2236	1.0247	1.526
.001800	.01696	.1066	.1062	.1068	1.0057	2.174	.007800	.03552	.2232	.2196	.2251	1.0250	1.521
.001900	.01743	.1095	.1091	.1097	1.0060	2.145	.007900	.03576	.2247	.2209	.2265	1.0253	1.517
.002000	.01788	.1123	.1119	.1125	1.0063	2.119	.008000	.03598	.2261	.2223	.2280	1.0257	1.512
.002100	.01832	.1151	.1146	.1154	1.0066	2.094	.008100	.03621	.2275	.2237	.2295	1.0260	1.508
.002200	.01876	.1178	.1173	.1181	1.0069	2.070	.008200	.03644	.2290	.2250	.2310	1.0263	1.503
.002300	.01918	.1205	.1199	.1208	1.0073	2.047	.008300	.03666	.2304	.2264	.2324	1.0266	1.499
.002400	.01959	.1231	.1225	.1234	1.0076	2.025	.008400	.03689	.2318	.2277	.2338	1.0270	1.495
.002500	.02000	.1257	.1250	.1260	1.0079	2.005	.008500	.03711	.2332	.2290	.2353	1.0273	1.491
.002600	.02040	.1282	.1275	.1285	1.0082	1.986	.008600	.03733	.2346	.2303	.2367	1.0276	1.487
.002700	.02079	.1306	.1299	.1310	1.0085	1.967	.008700	.03755	.2360	.2317	.2381	1.0280	1.482
.002800	.02117	.1330	.1323	.1334	1.0089	1.950	.008800	.03777	.2373	.2330	.2396	1.0283	1.478
.002900	.02155	.1354	.1346	.1358	1.0092	1.933	.008900	.03799	.2387	.2343	.2410	1.0286	1.474
.003000	.02192	.1377	.1369	.1382	1.0095	1.917	.009000	.03821	.2401	.2356	.2424	1.0290	1.471
.003100	.02228	.1400	.1391	.1405	1.0098	1.902	.009100	.03842	.2414	.2368	.2438	1.0293	1.467
.003200	.02264	.1423	.1413	.1427	1.0101	1.887	.009200	.03864	.2428	.2381	.2452	1.0296	1.463
.003300	.02300	.1445	.1435	.1449	1.0104	1.873	.009300	.03885	.2441	.2394	.2465	1.0299	1.459
.003400	.02335	.1467	.1456	.1472	1.0108	1.860	.009400	.03906	.2455	.2407	.2479	1.0303	1.456
.003500	.02369	.1488	.1477	.1494	1.0111	1.847	.009500	.03928	.2468	.2419	.2493	1.0306	1.452
.003600	.02403	.1510	.1498	.1515	1.0114	1.834	.009600	.03949	.2481	.2431	.2507	1.0309	1.448
.003700	.02436	.1531	.1519	.1537	1.0117	1.822	.009700	.03970	.2494	.2443	.2520	1.0313	1.445
.003800	.02469	.1551	.1539	.1558	1.0121	1.810	.009800	.03990	.2507	.2456	.2534	1.0316	1.442
.003900	.02502	.1572	.1559	.1579	1.0124	1.799	.009900	.04011	.2520	.2468	.2547	1.0319	1.438
.004000	.02534	.1592	.1579	.1599	1.0127	1.788	.01000	.04032	.2533	.2480	.2560	1.0322	1.435
.004100	.02566	.1612	.1598	.1619	1.0130	1.777	.01100	.04233	.2660	.2598	.2691	1.0356	1.403
.004200	.02597	.1632	.1617	.1639	1.0133	1.767	.01200	.04426	.2781	.2711	.2817	1.0389	1.375
.004300	.02628	.1651	.1636	.1659	1.0137	1.756	.01300	.04612	.2898	.2820	.2938	1.0423	1.350
.004400	.02659	.1671	.1655	.1678	1.0140	1.746	.01400	.04791	.3010	.2924	.3056	1.0456	1.327
.004500	.02689	.1690	.1674	.1698	1.0143	1.737	.01500	.04964	.3119	.3022	.3170	1.0490	1.307
.004600	.02719	.1708	.1692	.1717	1.0146	1.727	.01600	.05132	.3225	.3117	.3281	1.0524	1.288
.004700	.02749	.1727	.1710	.1736	1.0149	1.718	.01700	.05296	.3328	.3209	.3389	1.0559	1.271
.004800	.02778	.1745	.1728	.1754	1.0153	1.709	.01800	.05455	.3428	.3298	.3495	1.0593	1.255
.004900	.02807	.1764	.1746	.1773	1.0156	1.701	.01900	.05611	.3525	.3386	.3599	1.0628	1.240
.005000	.02836	.1782	.1764	.1791	1.0159	1.692	.02000	.05763	.3621	.3470	.3701	1.0663	1.226
.005100	.02864	.1800	.1781	.1809	1.0162	1.684	.02100	.05912	.3714	.3552	.3800	1.0698	1.213
.005200	.02893	.1818	.1798	.1827	1.0166	1.676	.02200	.06057	.3806	.3632	.3898	1.0733	1.201
.005300	.02921	.1835	.1815	.1845	1.0169	1.669	.02300	.06200	.3896	.3710	.3995	1.0768	1.189
.005400	.02948	.1852	.1832	.1863	1.0172	1.662	.02400	.06340	.3984	.3786	.4090	1.0804	1.178
.005500	.02976	.1870	.1848	.1880	1.0175	1.654	.02500	.06478	.4070	.3860	.4184	1.0840	1.168
.005600	.03003	.1887	.1865	.1898	1.0178	1.647	.02600	.06613	.4155	.3932	.4276	1.0876	1.159
.005700	.03030	.1904	.1881	.1915	1.0182	1.640	.02700	.06747	.4239	.4002	.4367	1.0912	1.150
.005800	.03057	.1921	.1897	.1932	1.0185	1.633	.02800	.06878	.4322	.4071	.4457	1.0949	1.141
.005900	.03083	.1937	.1913	.1949	1.0188	1.626	.02900	.07007	.4403	.4138	.4546	1.0985	1.133

d/L_o	d/L	$2\pi d/L$	TANH $2\pi d/L$	SINH $2\pi d/L$	COSH $2\pi d/L$	H/H'_o	d/L_o	d/L	$2\pi d/L$	TANH $2\pi d/L$	SINH $2\pi d/L$	COSH $2\pi d/L$	H/H'_o
.03000	.07135	.4483	.4205	.4634	1.1021	1.125	.09000	.1322	.8306	.6808	.9295	1.3653	.9422
.03100	.07260	.4562	.4269	.4721	1.1059	1.118	.09100	.1331	.8363	.6838	.9372	1.3706	.9411
.03200	.07385	.4640	.4333	.4808	1.1096	1.111	.09200	.1340	.8420	.6868	.9450	1.3759	.9401
.03300	.07507	.4717	.4395	.4894	1.1133	1.104	.09300	.1349	.8474	.6897	.9525	1.3810	.9391
.03400	.07630	.4794	.4457	.4980	1.1171	1.098	.09400	.1357	.8528	.6925	.9600	1.3862	.9381
.03500	.07748	.4868	.4517	.5064	1.1209	1.092	.09500	.1366	.8583	.6953	.9677	1.3917	.9371
.03600	.07867	.4943	.4577	.5147	1.1247	1.086	.09600	.1375	.8639	.6982	.9755	1.3970	.9362
.03700	.07984	.5017	.4635	.5230	1.1285	1.080	.09700	.1384	.8694	.7011	.9832	1.4023	.9353
.03800	.08100	.5090	.4691	.5312	1.1324	1.075	.09800	.1392	.8749	.7039	.9908	1.4077	.9344
.03900	.08215	.5162	.4747	.5394	1.1362	1.069	.09900	.1401	.8803	.7066	.9985	1.4131	.9335
.04000	.08329	.5233	.4802	.5475	1.1401	1.064	.1000	.1410	.8858	.7093	1.006	1.4187	.9327
.04100	.08442	.5304	.4857	.5556	1.1440	1.059	.1010	.1419	.8913	.7120	1.014	1.4242	.9319
.04200	.08553	.5374	.4911	.5637	1.1479	1.055	.1020	.1427	.8967	.7147	1.022	1.4297	.9311
.04300	.08664	.5444	.4964	.5717	1.1518	1.050	.1030	.1436	.9023	.7173	1.030	1.4354	.9304
.04400	.08774	.5513	.5015	.5796	1.1558	1.046	.1040	.1445	.9076	.7200	1.037	1.4410	.9297
.04500	.08883	.5581	.5066	.5876	1.1599	1.042	.1050	.1453	.9130	.7226	1.045	1.4465	.9290
.04600	.08991	.5649	.5116	.5954	1.1639	1.038	.1060	.1462	.9184	.7252	1.053	1.4523	.9282
.04700	.09098	.5717	.5166	.6033	1.1679	1.034	.1070	.1470	.9239	.7277	1.061	1.4580	.9276
.04800	.09205	.5784	.5215	.6111	1.1720	1.030	.1080	.1479	.9293	.7303	1.069	1.4638	.9269
.04900	.09311	.5850	.5263	.6189	1.1760	1.026	.1090	.1488	.9343	.7327	1.076	1.4692	.9263
.05000	.09416	.5916	.5310	.6267	1.1802	1.023	.1100	.1496	.9400	.7352	1.085	1.4752	.9257
.05100	.09520	.5981	.5357	.6344	1.1843	1.019	.1110	.1505	.9456	.7377	1.093	1.4814	.9251
.05200	.09623	.6046	.5403	.6421	1.1884	1.016	.1120	.1513	.9508	.7402	1.101	1.4871	.9245
.05300	.09726	.6111	.5449	.6499	1.1926	1.013	.1130	.1522	.9563	.7426	1.109	1.4932	.9239
.05400	.09829	.6176	.5494	.6575	1.1968	1.010	.1140	.1530	.9616	.7450	1.117	1.4990	.9234
.05500	.09930	.6239	.5538	.6652	1.2011	1.007	.1150	.1539	.9670	.7474	1.125	1.5051	.9228
.05600	.1003	.6303	.5582	.6729	1.2053	1.004	.1160	.1547	.9720	.7497	1.133	1.5108	.9223
.05700	.1013	.6366	.5626	.6805	1.2096	1.001	.1170	.1556	.9775	.7520	1.141	1.5171	.9218
.05800	.1023	.6428	.5668	.6880	1.2138	.9985	.1180	.1564	.9827	.7543	1.149	1.5230	.9214
.05900	.1033	.6491	.5711	.6956	1.2181	.9958	.1190	.1573	.9882	.7566	1.157	1.5293	.9209
.06000	.1043	.6553	.5753	.7033	1.2225	.9932	.1200	.1581	.9936	.7589	1.165	1.5356	.9204
.06100	.1053	.6616	.5794	.7110	1.2270	.9907	.1210	.1590	.9989	.7612	1.174	1.5418	.9200
.06200	.1063	.6678	.5834	.7187	1.2315	.9883	.1220	.1598	1.004	.7634	1.182	1.5479	.9196
.06300	.1073	.6739	.5874	.7256	1.2355	.9860	.1230	.1607	1.010	.7656	1.190	1.5546	.9192
.06400	.1082	.6799	.5914	.7335	1.2402	.9837	.1240	.1615	1.015	.7678	1.198	1.5605	.9189
.06500	.1092	.6860	.5954	.7411	1.2447	.9815	.1250	.1624	1.020	.7700	1.207	1.5674	.9186
.06600	.1101	.6920	.5993	.7486	1.2492	.9793	.1260	.1632	1.025	.7721	1.215	1.5734	.9182
.06700	.1111	.6981	.6031	.7561	1.2537	.9772	.1270	.1640	1.030	.7742	1.223	1.5795	.9178
.06800	.1120	.7037	.6069	.7633	1.2580	.9752	.1280	.1649	1.036	.7763	1.231	1.5862	.9175
.06900	.1130	.7099	.6106	.7711	1.2628	.9732	.1290	.1657	1.041	.7783	1.240	1.5927	.9172
.07000	.1139	.7157	.6144	.7783	1.2672	.9713	.1300	.1665	1.046	.7804	1.248	1.5990	.9169
.07100	.1149	.7219	.6181	.7863	1.2721	.9694	.1310	.1674	1.052	.7824	1.257	1.6060	.9166
.07200	.1158	.7277	.6217	.7937	1.2767	.9676	.1320	.1682	1.057	.7844	1.265	1.6124	.9164
.07300	.1168	.7336	.6252	.8011	1.2813	.9658	.1330	.1691	1.062	.7865	1.273	1.6191	.9161
.07400	.1177	.7395	.6289	.8088	1.2861	.9641	.1340	.1699	1.068	.7885	1.282	1.6260	.9158
.07500	.1186	.7453	.6324	.8162	1.2908	.9624	.1350	.1708	1.073	.7905	1.291	1.633	.9156
.07600	.1195	.7511	.6359	.8237	1.2956	.9607	.1360	.1716	1.078	.7925	1.300	1.640	.9154
.07700	.1205	.7569	.6392	.8312	1.3004	.9591	.1370	.1724	1.084	.7945	1.308	1.647	.9152
.07800	.1214	.7625	.6427	.8386	1.3051	.9576	.1380	.1733	1.089	.7964	1.317	1.654	.9150
.07900	.1223	.7683	.6460	.8462	1.3100	.9562	.1390	.1741	1.094	.7983	1.326	1.660	.9148
.08000	.1232	.7741	.6493	.8538	1.3149	.9548	.1400	.1749	1.099	.8002	1.334	1.667	.9146
.08100	.1241	.7799	.6526	.8614	1.3198	.9534	.1410	.1758	1.105	.8021	1.343	1.675	.9144
.08200	.1251	.7854	.6558	.8687	1.3246	.9520	.1420	.1766	1.110	.8039	1.352	1.681	.9142
.08300	.1259	.7911	.6590	.8762	1.3295	.9506	.1430	.1774	1.115	.8057	1.360	1.688	.9141
.08400	.1268	.7967	.6622	.8837	1.3345	.9493	.1440	.1783	1.120	.8076	1.369	1.696	.9140
.08500	.1277	.8026	.6655	.8915	1.3397	.9481	.1450	.1791	1.125	.8094	1.378	1.703	.9139
.08600	.1286	.8080	.6685	.8989	1.3446	.9469	.1460	.1800	1.131	.8112	1.388	1.710	.9137
.08700	.1295	.8137	.6716	.9064	1.3497	.9457	.1470	.1808	1.136	.8131	1.397	1.718	.9136
.08800	.1304	.8193	.6747	.9141	1.3548	.9445	.1480	.1816	1.141	.8149	1.405	1.725	.9135
.08900	.1313	.8250	.6778	.9218	1.3600	.9433	.1490	.1825	1.146	.8166	1.415	1.732	.9134

d/L_o	d/L	$2\pi\, d/L$	TANH $2\pi\, d/L$	SINH $2\pi\, d/L$	COSH $2\pi\, d/L$	H/H'_o
.1500	.1833	1.152	.8183	1.424	1.740	.9133
.1510	.1841	1.157	.8200	1.433	1.747	.9133
.1520	.1850	1.162	.8217	1.442	1.755	.9132
.1530	.1858	1.167	.8234	1.451	1.762	.9132
.1540	.1866	1.173	.8250	1.460	1.770	.9132
.1550	.1875	1.178	.8267	1.469	1.777	.9131
.1560	.1883	1.183	.8284	1.479	1.785	.9130
.1570	.1891	1.188	.8301	1.488	1.793	.9129
.1580	.1900	1.194	.8317	1.498	1.801	.9130
.1590	.1908	1.199	.8333	1.507	1.809	.9130
.1600	.1917	1.204	.8349	1.517	1.817	.9130
.1610	.1925	1.209	.8365	1.527	1.825	.9130
.1620	.1933	1.215	.8381	1.536	1.833	.9130
.1630	.1941	1.220	.8396	1.546	1.841	.9130
.1640	.1950	1.225	.8411	1.555	1.849	.9130
.1650	. 958	1.230	.8427	1.565	1.857	.9131
.1660	.[illegible]66	1.235	.8442	1.574	1.865	.9132
.1670	.1975	1.240	.8457	1.584	1.873	.9132
.1680	.1983	1.246	.8472	1.594	1.882	.9133
.1690	.1992	1.251	.8486	1.604	1.890	.9133
.1700	.2000	1.257	.8501	1.614	1.899	.9134
.1710	.2008	1.262	.8515	1.624	1.907	.9135
.1720	.2017	1.267	.8529	1.634	1.915	.9136
.1730	.2025	1.272	.8544	1.644	1.924	.9137
.1740	.2033	1.277	.8558	1.654	1.933	.9138
.1750	.2042	1.282	.8572	1.664	1.941	.9139
.1760	.2050	1.288	.8586	1.675	1.951	.9140
.1770	.2058	1.293	.8600	1.685	1.959	.9141
.1780	.2066	1.298	.8614	1.695	1.968	.9142
.1790	.2075	1.304	.8627	1.706	1.977	.9144
.1800	.2083	1.309	.8640	1.716	1.986	.9145
.1810	.2092	1.314	.8653	1.727	1.995	.9146
.1820	.2100	1.320	.8666	1.737	2.004	.9148
.1830	.2108	1.325	.8680	1.748	2.013	.9149
.1840	.2117	1.330	.8693	1.758	2.022	.9150
.1850	.2125	1.335	.8706	1.769	2.032	.9152
.1860	.2134	1.341	.8718	1.780	2.041	.9154
.1870	.2142	1.346	.8731	1.791	2.051	.9155
.1880	.2150	1.351	.8743	1.801	2.060	.9157
.1890	.2159	1.356	.8755	1.812	2.070	.9159
.1900	.2167	1.362	.8767	1.823	2.079	.9161
.1910	.2176	1.367	.8779	1.834	2.089	.9163
.1920	.2184	1.372	.8791	1.845	2.099	.9165
.1930	.2192	1.377	.8803	1.856	2.108	.9167
.1940	.2201	1.383	.8815	1.867	2.118	.9169
.1950	.2209	1.388	.8827	1.879	2.128	.9170
.1960	.2218	1.393	.8839	1.890	2.138	.9172
.1970	.2226	1.399	.8850	1.901	2.148	.9174
.1980	.2234	1.404	.8862	1.913	2.158	.9176
.1990	.2243	1.409	.8873	1.924	2.169	.9179
.2000	.2251	1.414	.8884	1.935	2.178	.9181
.2010	.2260	1.420	.8895	1.947	2.189	.9183
.2020	.2268	1.425	.8906	1.959	2.199	.9186
.2030	.2277	1.430	.8917	1.970	2.210	.9188
.2040	.2285	1.436	.8928	1.982	2.220	.9190
.2050	.2293	1.441	.8939	1.994	2.231	.9193
.2060	.2302	1.446	.8950	2.006	2.242	.9195
.2070	.2310	1.451	.8960	2.017	2.252	.9197
.2080	.2319	1.457	.8971	2.030	2.263	.9200
.2090	.2328	1.462	.8981	2.042	2.274	.9202

d/L_o	d/L	$2\pi\, d/L$	TANH $2\pi\, d/L$	SINH $2\pi\, d/L$	COSH $2\pi\, d/L$	H/H'_o
.2100	.2336	1.468	.8991	2.055	2.285	.9205
.2110	.2344	1.473	.9001	2.066	2.295	.9207
.2120	.2353	1.479	.9011	2.079	2.307	.9210
.2130	.2361	1.484	.9021	2.091	2.318	.9213
.2140	.2370	1.489	.9031	2.103	2.329	.9215
.2150	.2378	1.494	.9041	2.115	2.340	.9218
.2160	.2387	1.500	.9051	2.128	2.351	.9221
.2170	.2395	1.506	.9061	2.142	2.364	.9223
.2180	.2404	1.511	.9070	2.154	2.375	.9226
.2190	.2412	1.516	.9079	2.166	2.386	.9228
.2200	.2421	1.521	.9088	2.178	2.397	.9231
.2210	.2429	1.526	.9097	2.192	2.409	.9234
.2220	.2438	1.532	.9107	2.204	2.421	.9236
.2230	.2446	1.537	.9116	2.218	2.433	.9239
.2240	.2455	1.542	.9125	2.230	2.444	.9242
.2250	.2463	1.548	.9134	2.244	2.457	.9245
.2260	.2472	1.553	.9143	2.257	2.469	.9248
.2270	.2481	1.559	.9152	2.271	2.481	.9251
.2280	.2489	1.564	.9161	2.284	2.493	.9254
.2290	.2498	1.569	.9170	2.297	2.506	.9258
.2300	.2506	1.575	.9178	2.311	2.518	.9261
.2310	.2515	1.580	.9186	2.325	2.531	.9264
.2320	.2523	1.585	.9194	2.338	2.543	.9267
.2330	.2532	1.591	.9203	2.352	2.556	.9270
.2340	.2540	1.596	.9211	2.366	2.569	.9273
.2350	.2549	1.602	.9219	2.38[illegible]	2.581	.9276
.2360	.2558	1.607	.9227	2.39[illegible]	2.594	.9279
.2370	.2566	1.612	.9235	2.408	2.607	.9282
.2380	.2575	1.618	.9243	2.422	2.620	.9285
.2390	.2584	1.623	.9251	2.436	2.634	.9288
.2400	.2592	1.629	.9259	2.450	2.647	.9291
.2410	.2601	1.634	.9267	2.464	2.660	.9294
.2420	.2610	1.640	.9275	2.480	2.674	.9298
.2430	.2618	1.645	.9282	2.494	2.687	.9301
.2440	.2627	1.650	.9289	2.508	2.700	.9304
.2450	.2635	1.656	.9296	2.523	2.714	.9307
.2460	.2644	1.661	.9304	2.538	2.728	.9310
.2470	.2653	1.667	.9311	2.553	2.742	.9314
.2480	.2661	1.672	.9318	2.568	2.755	.9317
.2490	.2670	1.678	.9325	2.583	2.770	.9320
.2500	.2679	1.683	.9332	2.599	2.784	.9323
.2510	.2687	1.689	.9339	2.614	2.798	.9327
.2520	.2696	1.694	.9346	2.629	2.813	.9330
.2530	.2705	1.700	.9353	2.645	2.828	.9333
.2540	.2714	1.705	.9360	2.660	2.842	.9336
.2550	.2722	1.711	.9367	2.676	2.856	.9340
.2560	.2731	1.716	.9374	2.691	2.871	.9343
.2570	.2740	1.722	.9381	2.707	2.886	.9346
.2580	.2749	1.727	.9388	2.723	2.901	.9349
.2590	.2757	1.732	.9394	2.739	2.916	.9353
.2600	.2766	1.738	.9400	2.755	2.931	.9356
.2610	.2775	1.744	.9406	2.772	2.946	.9360
.2620	.2784	1.749	.9412	2.788	2.962	.9363
.2630	.2792	1.755	.9418	2.804	2.977	.9367
.2640	.2801	1.760	.9425	2.820	2.992	.9370
.2650	.2810	1.766	.9431	2.837	3.008	.9373
.2660	.2819	1.771	.9437	2.853	3.023	.9377
.2670	.2827	1.776	.9443	2.870	3.039	.9380
.2680	.2836	1.782	.9449	2.886	3.055	.9383
.2690	.2845	1.788	.9455	2.904	3.071	.9386

d/L_o	d/L	$2\pi d/L$	TANH $2\pi d/L$	SINH $2\pi d/L$	COSH $2\pi d/L$	H/H_o'
.2700	.2854	1.793	.9461	2.921	3.088	.9390
.2710	.2863	1.799	.9467	2.938	3.104	.9393
.2720	.2872	1.804	.9473	2.956	3.120	.9396
.2730	.2880	1.810	.9478	2.973	3.136	.9400
.2740	.2889	1.815	.9484	2.990	3.153	.9403
.2750	.2898	1.821	.9490	3.008	3.170	.9406
.2760	.2907	1.826	.9495	3.025	3.186	.9410
.2770	.2916	1.832	.9500	3.043	3.203	.9413
.2780	.2924	1.837	.9505	3.061	3.220	.9416
.2790	.2933	1.843	.9511	3.079	3.237	.9420
.2800	.2942	1.849	.9516	3.097	3.254	.9423
.2810	.2951	1.854	.9521	3.115	3.272	.9426
.2820	.2960	1.860	.9526	3.133	3.289	.9430
.2830	.2969	1.866	.9532	3.152	3.307	.9433
.2840	.2978	1.871	.9537	3.171	3.325	.9436
.2850	.2987	1.877	.9542	3.190	3.343	.9440
.2860	.2996	1.882	.9547	3.209	3.361	.9443
.2870	.3005	1.888	.9552	3.228	3.379	.9446
.2880	.3014	1.893	.9557	3.246	3.396	.9449
.2890	.3022	1.899	.9562	3.264	3.414	.9452
.2900	.3031	1.905	.9567	3.284	3.433	.9456
.2910	.3040	1.910	.9572	3.303	3.451	.9459
.2920	.3049	1.916	.9577	3.323	3.471	.9463
.2930	.3058	1.922	.9581	3.343	3.49[illegible]	.9466
.2940	.3067	1.927	.9585	3.362	3.[illegible]08	.9469
.2950	.3076	1.933	.9590	3.382	3.527	.9473
.2960	.3085	1.938	.9594	3.402	3.546	.9476
.2970	.3094	1.944	.9599	3.422	3.565	.9480
.2980	.3103	1.950	.9603	3.442	3.585	.9483
.2990	.3112	1.955	.9607	3.462	3.604	.9486
.3000	.3121	1.961	.9611	3.483	3.624	.9490
.3010	.3130	1.967	.9616	3.503	3.643	.9493
.3020	.3139	1.972	.9620	3.524	3.663	.9496
.3030	.3148	1.978	.9624	3.545	3.683	.9499
.3040	.3157	1.984	.9629	3.566	3.703	.9502
.3050	.3166	1.989	.9633	3.587	3.724	.9505
.3060	.3175	1.995	.9637	3.609	3.745	.9509
.3070	.3184	2.001	.9641	3.630	3.765	.9512
.3080	.3193	2.007	.9645	3.651	3.786	.9515
.3090	.3202	2.012	.9649	3.673	3.806	.9518
.3100	.3211	2.018	.9653	3.694	3.827	.9522
.3110	.3220	2.023	.9656	3.716	3.848	.9525
.3120	.3230	2.029	.9660	3.738	3.870	.9528
.3130	.3239	2.035	.9664	3.760	3.891	.9531
.3140	.3248	2.041	.9668	3.782	3.912	.9535
.3150	.3257	2.046	.9672	3.805	3.934	.9538
.3160	.3266	2.052	.9676	3.828	3.956	.9541
.3170	.3275	2.058	.9679	3.851	3.978	.9544
.3180	.3284	2.063	.9682	3.873	4.000	.9547
.3190	.3294	2.069	.9686	3.896	4.022	.9550
.3200	.3302	2.075	.9690	3.919	4.045	.9553
.3210	.3311	2.081	.9693	3.943	4.068	.9556
.3220	.3321	2.086	.9696	3.966	4.090	.9559
.3230	.3330	2.092	.9700	3.990	4.114	.9562
.3240	.3339	2.098	.9703	4.014	4.136	.9565
.3250	.3349	2.104	.9707	4.038	4.160	.9568
.3260	.3357	2.110	.9710	4.061	4.183	.9571
.3270	.3367	2.115	.9713	4.085	4.206	.9574
.3280	.3376	2.121	.9717	4.110	4.230	.9577
.3290	.3385	2.127	.9720	4.135	4.254	.9580

d/L_o	d/L	$2\pi d/L$	TANH $2\pi d/L$	SINH $2\pi d/L$	COSH $2\pi d/L$	H/H_o'
.3300	.3394	2.133	.9723	4.159	4.277	.9583
.3310	.3403	2.138	.9726	4.184	4.301	.9586
.3320	.3413	2.144	.9729	4.209	4.326	.9589
.3330	.3422	2.150	.9732	4.234	4.350	.9592
.3340	.3431	2.156	.9735	4.259	4.375	.9595
.3350	.3440	2.161	.9738	4.284	4.399	.9598
.3360	.3449	2.167	.9741	4.310	4.424	.9601
.3370	.3459	2.173	.9744	4.336	4.450	.9604
.3380	.3468	2.179	.9747	4.361	4.474	.9607
.3390	.3477	2.185	.9750	4.388	4.500	.9610
.3400	.3468	2.190	.9753	4.413	4.525	.9613
.3410	.3495	2.196	.9756	4.439	4.550	.9615
.3420	.3504	2.202	.9758	4.466	4.576	.9618
.3430	.3514	2.208	.9761	4.492	4.602	.9621
.3440	.3523	2.214	.9764	4.521	4.630	.9623
.3450	.3532	2.220	.9767	4.547	4.656	.9626
.3460	.3542	2.225	.9769	4.575	4.682	.9629
.3470	.3551	2.231	.9772	4.602	4.709	.9632
.3480	.3560	2.237	.9775	4.629	4.736	.9635
.3490	.3570	2.243	.9777	4.657	4.763	.9638
.3500	.3579	2.249	.9780	4.685	4.791	.9640
.3510	.3588	2.255	.9782	4.713	4.818	.9643
.3520	.3598	2.260	.9785	4.741	4.845	.9646
.3530	.3607	2.266	.9787	4.770	4.873	.9648
.3540	.3616	2.272	.9790	4.798	4.901	.9651
.3550	.3625	2.278	.9792	4.827	4.929	.9654
.3560	.3635	2.284	.9795	4.856	4.957	.9657
.3570	.3644	2.290	.9797	4.885	4.987	.9659
.3580	.3653	2.296	.9799	4.914	5.015	.9662
.3590	.3663	2.301	.9801	4.944	5.044	.9665
.3600	.3672	2.307	.9804	4.974	5.072	.9667
.3610	.3682	2.313	.9806	5.004	5.103	.9670
.3620	.3691	2.319	.9808	5.034	5.132	.9673
.3630	.3700	2.325	.9811	5.063	5.161	.9675
.3640	.3709	2.331	.9813	5.094	5.191	.9677
.3650	.3719	2.337	.9815	5.124	5.221	.9680
.3660	.3728	2.342	.9817	5.155	5.251	.9683
.3670	.3737	2.348	.9819	5.186	5.281	.9686
.3680	.3747	2.354	.9821	5.217	5.312	.9688
.3690	.3756	2.360	.9823	5.248	5.343	.9690
.3700	.3766	2.366	.9825	5.280	5.374	.9693
.3710	.3775	2.372	.9827	5.312	5.406	.9696
.3720	.3785	2.378	.9830	5.345	5.438	.9698
.3730	.3794	2.384	.9832	5.377	5.469	.9700
.3740	.3804	2.390	.9834	5.410	5.502	.9702
.3750	.3813	2.396	.9835	5.443	5.534	.9705
.3760	.3822	2.402	.9837	5.475	5.566	.9707
.3770	.3832	2.408	.9839	5.508	5.598	.9709
.3780	.3841	2.413	.9841	5.541	5.631	.9712
.3790	.3850	2.419	.9843	5.572	5.661	.9714
.3800	.3860	2.425	.9845	5.609	5.697	.9717
.3810	.3869	2.431	.9847	5.643	5.731	.9719
.3820	.3879	2.437	.9848	5.677	5.765	.9721
.3830	.3888	2.443	.9850	5.712	5.798	.9724
.3840	.3898	2.449	.9852	5.746	5.833	.9726
.3850	.3907	2.455	.9854	5.780	5.866	.9728
.3860	.3917	2.461	.9855	5.814	5.900	.9730
.3870	.3926	2.467	.9857	5.850	5.935	.9732
.3880	.3936	2.473	.9859	5.886	5.970	.9735
.3890	.3945	2.479	.9860	5.921	6.005	.9737

d/L_o	d/L	$2\pi d/L$	TANH $2\pi d/L$	SINH $2\pi d/L$	COSH $2\pi d/L$	H/H_o'
.3900	.3955	2.485	.9862	5.957	6.040	.9739
.3910	.3964	2.491	.9864	5.993	6.076	.9741
.3920	.3974	2.497	.9865	6.029	6.112	.9743
.3930	.3983	2.503	.9867	6.066	6.148	.9745
.3940	.3993	2.509	.9869	6.103	6.185	.9748
.3950	.4002	2.515	.9870	6.140	6.221	.9750
.3960	.4012	2.521	.9872	6.177	6.258	.9752
.3970	.4021	2.527	.9873	6.215	6.295	.9754
.3980	.4031	2.532	.9874	6.252	6.332	.9756
.3990	.4040	2.538	.9876	6.290	6.369	.9758
.4000	.4050	2.544	.9877	6.329	6.407	.9761
.4010	.4059	2.550	.9879	6.367	6.445	.9763
.4020	.4069	2.556	.9880	6.406	6.483	.9765
.4030	.4078	2.562	.9882	6.444	6.521	.9766
.4040	.4088	2.568	.9883	6.484	6.561	.9768
.4050	.4098	2.575	.9885	6.525	6.601	.9770
.4060	.4107	2.581	.9886	6.564	6.640	.9772
.4070	.4116	2.586	.9887	6.603	6.679	.9774
.4080	.4126	2.592	.9889	6.644	6.718	.9776
.4090	.4136	2.598	.9890	6.684	6.758	.9778
.4100	.4145	2.604	.9891	6.725	6.799	.9780
.4110	.4155	2.610	.9892	6.766	6.839	.9782
.4120	.4164	2.616	.9894	6.806	6.879	.9784
.4130	.4174	[illegible]	.9895	6.849	6.921	.9786
.4140	.4183	[illegible]	.9896	[illegible]	6.963	.9788
.4150	.4193	2.635	.9898	6.932	7.004	.9790
.4160	.4203	2.641	.9899	6.974	7.046	.9792
.4170	.4212	2.647	.9900	7.018	7.088	.9794
.4180	.4222	2.653	.9901	7.060	7.130	.9795
.4190	.4231	2.659	.9902	7.102	7.173	.9797
.4200	.4241	2.665	.9904	7.146	7.215	.9798
.4210	.4251	2.671	.9905	7.190	7.259	.9800
.4220	.4260	2.677	.9906	7.234	7.303	.9802
.4230	.4270	2.683	.9907	7.279	7.349	.9804
.4240	.4280	2.689	.9908	7.325	7.392	.9806
.4250	.4289	2.695	.9909	7.371	7.438	.9808
.4260	.4298	2.701	.9910	7.412	7.479	.9810
.4270	.4308	2.707	.9911	7.457	7.524	.9811
.4280	.4318	2.713	.9912	7.503	7.570	.9812
.4290	.4328	2.719	.9913	7.550	7.616	.9814
.4300	.4337	2.725	.9914	7.595	7.661	.9816
.4310	.4347	2.731	.9915	7.642	7.707	.9818
.4320	.4356	2.737	.9916	7.688	7.753	.9819
.4330	.4366	2.743	.9917	7.735	7.800	.9821
.4340	.4376	2.749	.9918	7.783	7.847	.9823
.4350	.4385	2.755	.9919	7.831	7.895	.9824
.4360	.4395	2.762	.9920	7.880	7.943	.9826
.4370	.4405	2.768	.9921	7.922	7.991	.9828
.4380	.4414	2.774	.9922	7.975	8.035	.9829
.4390	.4424	2.780	.9923	8.026	8.088	.9830
.4400	.4434	2.786	.9924	8.075	8.136	.9832
.4410	.4443	2.792	.9925	8.124	8.185	.9833
.4420	.4453	2.798	.9926	8.175	8.236	.9835
.4430	.4463	2.804	.9927	8.228	8.285	.9836
.4440	.4472	2.810	.9928	8.274	8.334	.9838
.4450	.4482	2.816	.9929	8.326	8.387	.9839
.4460	.4492	2.822	.9930	8.379	8.438	.9841
.4470	.4501	2.828	.9930	8.427	8.486	.9843
.4480	.4511	2.834	.9931	8.481	8.540	.9844
.4490	.4521	2.840	.9932	8.532	8.590	.9846

d/L_o	d/L	$2\pi d/L$	TANH $2\pi d/L$	SINH $2\pi d/L$	COSH 2 d/L	H/H_o'
.4500	.4531	2.847	.9933	8.585	8.643	.9847
.4510	.4540	2.853	.9934	8.638	8.695	.9848
.4520	.4550	2.859	.9935	8.693	8.750	.9849
.4530	.4560	2.865	.9935	8.747	8.804	.9851
.4540	.4569	2.871	.9936	8.797	8.854	.9852
.4550	.4579	2.877	.9937	8.853	8.910	.9853
.4560	.4589	2.883	.9938	8.910	8.965	.9855
.4570	.4599	2.890	.9938	8.965	9.021	.9857
.4580	.4608	2.896	.9939	9.016	9.072	.9858
.4590	.4618	2.902	.9940	9.074	9.129	.9859
.4600	.4628	2.908	.9941	9.132	9.186	.9860
.4610	.4637	2.914	.9941	9.183	9.238	.9862
.4620	.4647	2.920	.9942	9.242	9.296	.9863
.4630	.4657	2.926	.9943	9.301	9.354	.9864
.4640	.4666	2.932	.9944	9.353	9.406	.9865
.4650	.4676	2.938	.9944	9.413	9.466	.9867
.4660	.4686	2.944	.9945	9.472	9.525	.9868
.4670	.4695	2.951	.9946	9.533	9.585	.9869
.4680	.4705	2.957	.9946	9.586	9.638	.9871
.4690	.4715	2.963	.9947	9.647	9.699	.9872
.4700	.4725	2.969	.9947	9.709	9.760	.9873
.4710	.4735	2.975	.9948	9.770	9.821	.9874
.4720	.4744	2.981	.9949	9.826	9.877	.9875
.4730	.4754	2.987	.9949	9.888	9.938	.9876
.4740	.4764	2.993	.9950	9.951	10.00	.9877
.4750	.4774	2.999	.9951	10.01	10.07	.9878
.4760	.4783	3.005	.9951	10.07	10.12	.9880
.4770	.4793	3.012	.9952	10.13	10.18	.9881
.4780	.4803	3.018	.9952	10.20	10.25	.9882
.4790	.4813	3.024	.9953	10.26	10.31	.9883
.4800	.4822	3.030	.9953	10.32	10.37	.9885
.4810	.4832	3.036	.9954	10.39	10.43	.9886
.4820	.4842	3.042	.9955	10.45	10.50	.9887
.4830	.4852	3.049	.9955	10.52	10.57	.9888
.4840	.4862	3.055	.9956	10.59	10.63	.9889
.4850	.4871	3.061	.9956	10.65	10.69	.9890
.4860	.4881	3.067	.9957	10.71	10.76	.9891
.4870	.4891	3.073	.9957	10.78	10.83	.9892
.4880	.4901	3.079	.9958	10.85	10.90	.9893
.4890	.4911	3.086	.9958	10.92	10.96	.9895
.4900	.4920	3.092	.9959	10.99	11.03	.9896
.4910	.4930	3.098	.9959	11.05	11.09	.9897
.4920	.4940	3.104	.9960	11.12	11.16	.9898
.4930	.4950	3.110	.9960	11.19	11.24	.9899
.4940	.4960	3.117	.9961	11.26	11.31	.9899
.4950	.4969	3.122	.9961	11.32	11.37	.9900
.4960	.4979	3.128	.9962	11.40	11.44	.9901
.4970	.4989	3.135	.9962	11.47	11.51	.9902
.4980	.4999	3.141	.9963	11.54	11.59	.9903
.4990	.5009	3.147	.9963	11.61	11.65	.9904
.5000	.5018	3.153	.9964	11.68	11.72	.9905
.5010	.5028	3.159	.9964	11.75	11.80	.9906
.5020	.5038	3.166	.9964	11.83	11.87	.9907
.5030	.5048	3.172	.9965	11.91	11.95	.9908
.5040	.5058	3.178	.9965	11.98	12.02	.9909
.5050	.5067	3.184	.9966	12.05	12.09	.9909
.5060	.5077	3.190	.9966	12.12	12.16	.9910
.5070	.5087	3.196	.9967	12.20	12.24	.9911
.5080	.5097	3.203	.9967	12.28	12.32	.9912
.5090	.5107	3.209	.9968	12.35	12.39	.9913

d/L_o	d/L	$2\pi d/L$	TANH $2\pi d/L$	SINH $2\pi d/L$	COSH $2\pi d/L$	H/H'_o	d/L_o	d/L	$2\pi d/L$	TANH $2\pi d/L$	SINH $2\pi d/L$	COSH $2\pi d/L$	H/H'_o
.5100	.5117	3.215	.9968	12.43	12.47	.9914	.5700	.5709	3.587	.9985	18.05	18.08	.9953
.5110	.5126	3.221	.9968	12.50	12.54	.9915	.5710	.5719	3.593	.9985	18.16	18.19	.9953
.5120	.5136	3.227	.9969	12.58	12.62	.9915	.5720	.5729	3.600	.9985	18.28	18.31	.9954
.5130	.5146	3.233	.9969	12.66	12.70	.9916	.5730	.5738	3.606	.9985	18.39	18.42	.9954
.5140	.5156	3.240	.9970	12.74	12.78	.9917	.5740	.5748	3.612	.9985	18.50	18.53	.9955
.5150	.5166	3.246	.9970	12.82	12.86	.9918	.5750	.5758	3.618	.9986	18.62	18.64	.9955
.5160	.5176	3.252	.9970	12.90	12.94	.9919	.5760	.5768	3.624	.9986	18.73	18.76	.9956
.5170	.5185	3.258	.9971	12.98	13.02	.9919	.5770	.5778	3.630	.9986	18.85	18.88	.9956
.5180	.5195	3.264	.9971	13.06	13.10	.9920	.5780	.5788	3.637	.9986	18.97	19.00	.9957
.5190	.5205	3.270	.9971	13.14	13.18	.9921	.5790	.5798	3.643	.9986	19.09	19.12	.9957
.5200	.5215	3.277	.9972	13.22	13.26	.9922	.5800	.5808	3.649	.9987	19.21	19.24	.9957
.5210	.5225	3.283	.9972	13.31	13.35	.9923	.5810	.5818	3.656	.9987	19.33	19.36	.9958
.5220	.5235	3.289	.9972	13.39	13.43	.9924	.5820	.5828	3.662	.9987	19.45	19.48	.9958
.5230	.5244	3.295	.9973	13.47	13.51	.9924	.5830	.5838	3.668	.9987	19.58	19.60	.9959
.5240	.5254	3.301	.9973	13.55	13.59	.9925	.5840	.5848	3.674	.9987	19.70	19.73	.9959
.5250	.5264	3.308	.9973	13.64	13.68	.9926	.5850	.5858	3.680	.9987	19.81	19.84	.9960
.5260	.5274	3.314	.9974	13.73	13.76	.9927	.5860	.5867	3.686	.9987	19.94	19.96	.9960
.5270	.5284	3.320	.9974	13.81	13.85	.9927	.5870	.5877	3.693	.9988	20.06	20.09	.9960
.5280	.5294	3.326	.9974	13.90	13.94	.9928	.5880	.5887	3.699	.9988	20.19	20.21	.9961
.5290	.5304	3.333	.9975	13.99	14.02	.9929	.5890	.5897	3.705	.9988	20.32	20.34	.9961
.5300	.5314	3.339	.9975	14.07	14.10	.9930	.5900	.5907	3.712	.9988	20.45	20.47	.9962
.5310	.5323	3.345	.9975	14.16	14.19	.9931	.5910	.5917	3.718	.9988	20.57	20.60	.9962
.5320	[illegible]	3.351	.9976	14.25	14.28	.9931	.5920	.5927	3.724	.9988	20.70	20.73	.9963
.5330	.5343	3.357	.9976	14.34	14.37	.9932	.5930	.5937	3.730	.9989	20.83	20.86	.9963
.5340	.5353	3.363	.9976	14.43	14.46	.9933	.5940	.5947	3.737	.9989	20.97	20.99	.9963
.5350	.5363	3.370	.9976	14.52	14.55	.9933	.5950	.5957	3.743	.9989	21.10	21.12	.9964
.5360	.5373	3.376	.9977	14.61	14.64	.9934	.5960	.5967	3.749	.9989	21.23	21.25	.9964
.5370	.5383	3.382	.9977	14.70	14.73	.9935	.5970	.5977	3.755	.9989	21.35	21.37	.9964
.5380	.5393	3.388	.9977	14.79	14.82	.9935	.5980	.5987	3.761	.9989	21.49	21.51	.9965
.5390	.5402	3.394	.9977	14.88	14.91	.9936	.5990	.5996	3.767	.9989	21.62	21.64	.9965
.5400	.5412	3.401	.9978	14.97	15.01	.9936	.6000	.6006	3.774	.9990	21.76	21.78	.9965
.5410	.5422	3.407	.9978	15.07	15.10	.9937	.6100	.6106	3.836	.9991	23.17	23.19	.9969
.5420	.5432	3.413	.9978	15.16	15.19	.9938	.6200	.6205	3.899	.9992	24.66	24.68	.9972
.5430	.5442	3.419	.9979	15.25	15.29	.9938	.6300	.6305	3.961	.9993	26.25	26.27	.9975
.5440	.5452	3.426	.9979	15.35	15.38	.9939	.6400	.6404	4.024	.9994	27.95	27.97	.9977
.5450	.5461	3.432	.9979	15.45	15.48	.9940	.6500	.6504	4.086	.9994	29.75	29.77	.9980
.5460	.5471	3.438	.9979	15.54	15.58	.9941	.6600	.6603	4.149	.9995	31.68	31.69	.9982
.5470	.5481	3.444	.9980	15.64	15.67	.9941	.6700	.6703	4.212	.9996	33.73	33.74	.9983
.5480	.5491	3.450	.9980	15.74	15.77	.9942	.6800	.6803	4.274	.9996	35.90	35.92	.9985
.5490	.5501	3.456	.9980	15.84	15.87	.9942	.6900	.6902	4.337	.9997	38.23	38.24	.9987
.5500	.5511	3.463	.9980	15.94	15.97	.9942	.7000	.7002	4.400	.9997	40.71	40.72	.9988
.5510	.5521	3.469	.9981	16.04	16.07	.9942	.7100	.7102	4.462	.9997	43.34	43.35	.9989
.5520	.5531	3.475	.9981	16.14	16.17	.9943	.7200	.7202	4.525	.9998	46.14	46.15	.9990
.5530	.5541	3.481	.9981	16.24	16.27	.9944	.7300	.7302	4.588	.9998	49.13	49.14	.9991
.5540	.5551	3.488	.9981	16.34	16.37	.9944	.7400	.7401	4.650	.9998	52.31	52.32	.9992
.5550	.5560	3.494	.9982	16.44	16.47	.9945	.7500	.7501	4.713	.9998	55.70	55.71	.9993
.5560	.5570	3.500	.9982	16.54	16.57	.9945	.7600	.7601	4.776	.9999	59.31	59.31	.9994
.5570	.5580	3.506	.9982	16.65	16.68	.9946	.7700	.7701	4.839	.9999	63.15	63.16	.9995
.5580	.5590	3.512	.9982	16.75	16.78	.9947	.7800	.7801	4.902	.9999	67.24	67.25	.9996
.5590	.5600	3.519	.9982	16.85	16.88	.9947	.7900	.7901	4.964	.9999	71.60	71.60	.9996
.5600	.5610	3.525	.9983	16.96	16.99	.9947	.8000	.8001	5.027	.9999	76.24	76.24	.9996
.5610	.5620	3.531	.9983	17.06	17.09	.9948	.8100	.8101	5.090	.9999	81.18	81.19	.9996
.5620	.5630	3.537	.9983	17.17	17.20	.9949	.8200	.8201	5.153	.9999	86.44	86.44	.9997
.5630	.5640	3.543	.9983	17.28	17.31	.9949	.8300	.8301	5.215	.9999	92.04	92.05	.9997
.5640	.5649	3.550	.9984	17.38	17.41	.9950	.8400	.8400	5.278	1.000	98.00	98.01	.9997
.5650	.5659	3.556	.9984	17.49	17.52	.9950	.8500	.8500	5.341	1.000	104.4	104.4	.9998
.5660	.5669	3.562	.9984	17.60	17.63	.9951	.8600	.8600	5.404	1.000	111.1	111.1	.9998
.5670	.5679	3.568	.9984	17.71	17.74	.9951	.8700	.8700	5.467	1.000	118.3	118.3	.9998
.5680	.5689	3.575	.9984	17.82	17.85	.9952	.8800	.8800	5.529	1.000	126.0	126.0	.9998
.5690	.5699	3.581	.9985	17.94	17.97	.9952	.8900	.8900	5.592	1.000	134.2	134.2	.9998

d/L_o	d/L	$2\pi\, d/L$	TANH $2\pi\, d/L$	SINH $2\pi\, d/L$	COSH $2\pi\, d/L$	H/H_o'
.9000	.9000	5.655	1.000	142.9	142.9	.9999
.9100	.9100	5.718	1.000	152.1	152.1	.9999
.9200	.9200	5.781	1.000	162.0	162.0	.9999
.9300	.9300	5.844	1.000	172.5	172.5	.9999
.9400	.9400	5.906	1.000	183.7	183.7	.9999
.9500	.9500	5.969	1.000	195.6	195.6	.9999
.9600	.9600	6.032	1.000	208.2	208.2	.9999
.9700	.9700	6.095	1.000	221.7	221.7	.9999
.9800	.9800	6.158	1.000	236.1	236.1	.9999
.9900	.9900	6.220	1.000	251.4	251.4	1.000
1.000	1.000	6.283	1.000	267.7	267.7	1.000

Appendix D

SELECTED TABLES AND FIGURES USED IN THE DESIGN OF CABLE PROTECTION SYSTEMS

Table 5-1. Weight and Density of Typical Cable Stabilization System Components

Component	Weight Per Unit Length		Density, ρ_i (lb/ft^3)
	In Air, W^* (lb/ft)	In Water, W_s^* (lb/ft)	
Cables			
SDC List 3	5.27	3.56	194
SDC List 4	7.28	5.27	226
SDC List 5	14.75	11.45	280
Stabilization System Components			
Concrete	–	–	160
Split-Pipe			
3-1/2-in. ID	43	40	450
5-in. ID	60.4	57.2	450
Chain (stud link)			
2 in.	39.2	34	485
2-1/2 in.	61.4	53.3	485
3 in.	89.3	77.5	485
Chain (close link)			
2 in.	40	34.7	485
2-1/2 in.	65	56.4	485
3 in.	86	74.6	485

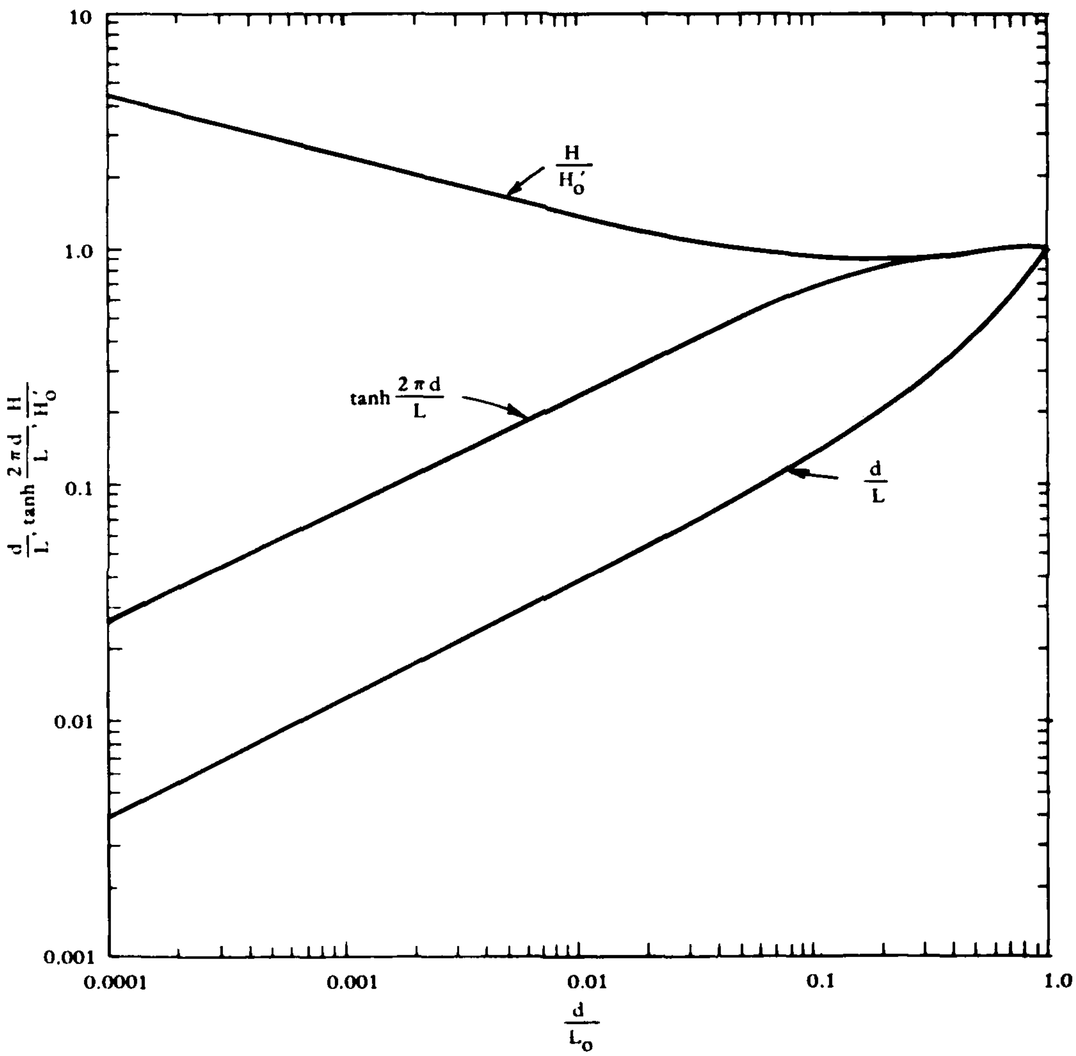


Figure 5-4. Value of various parameters as a function of d/L_o.

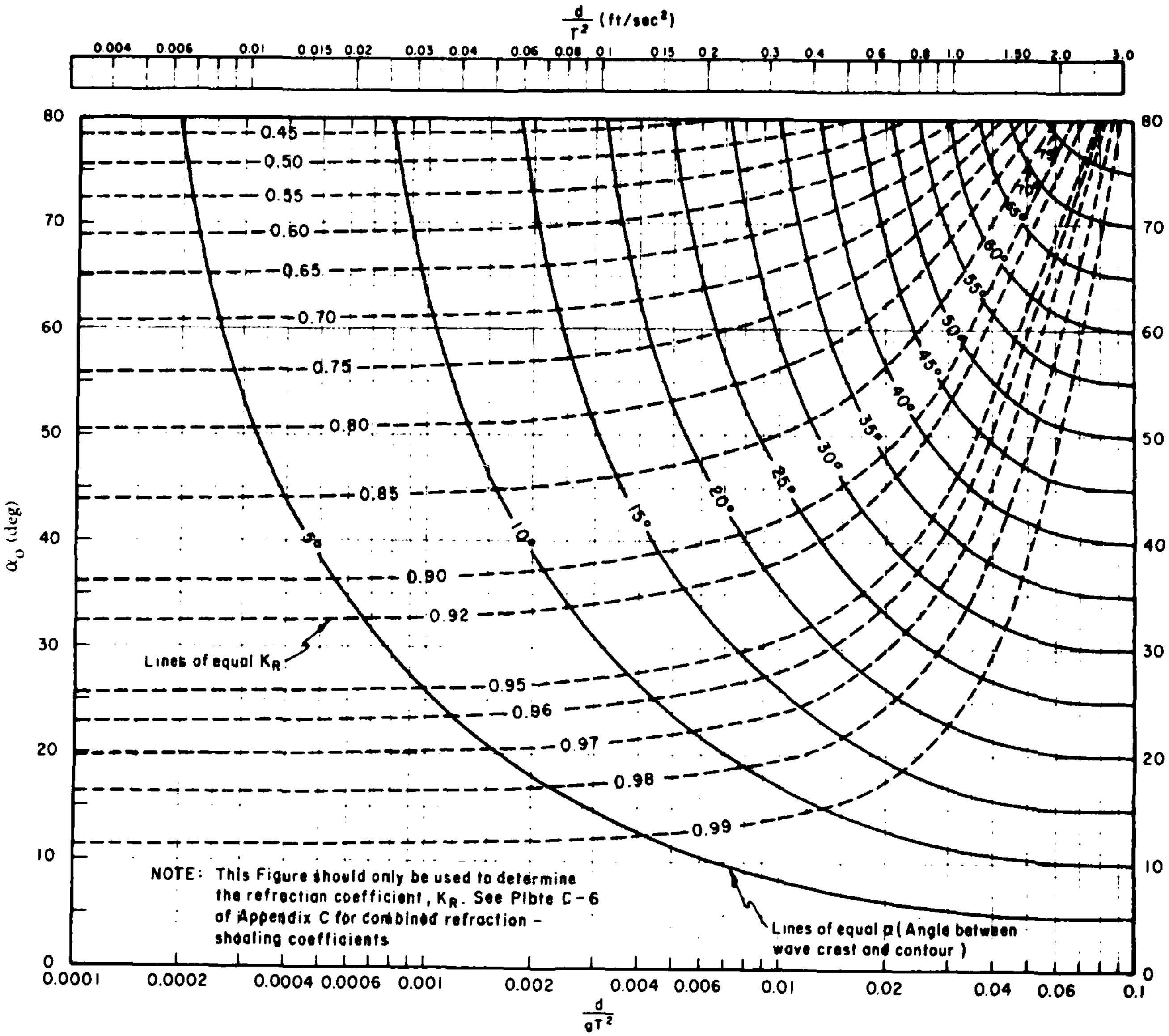


Figure 5-5. Changes in wave direction and height due to refraction on slopes with straight, parallel depth contours (from Shore Protection Manual).

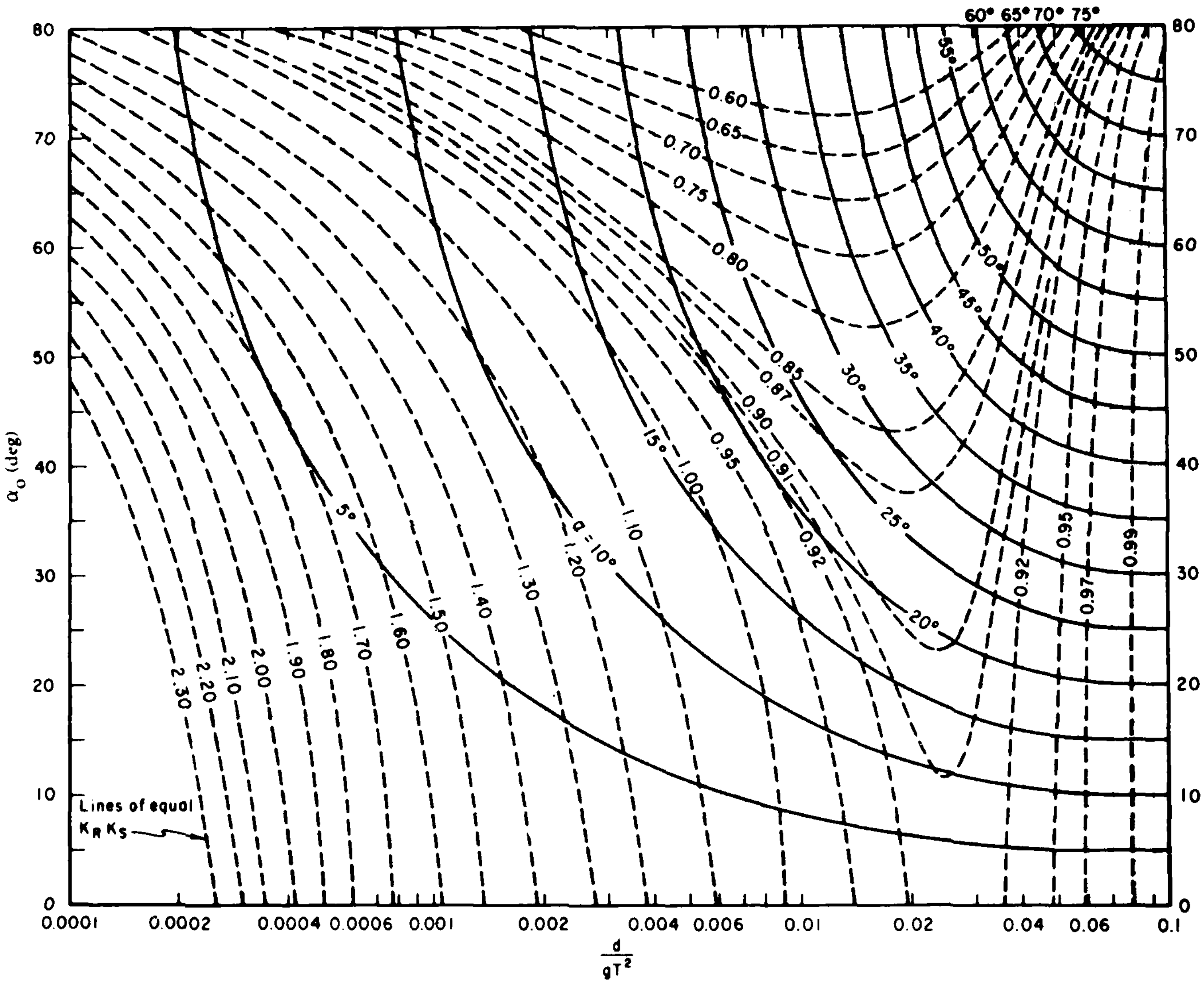


Figure 5-6. Change in wave direction and height due to refraction on slopes with straight, parallel depth contours including shoaling (from Shore Protection Manual).

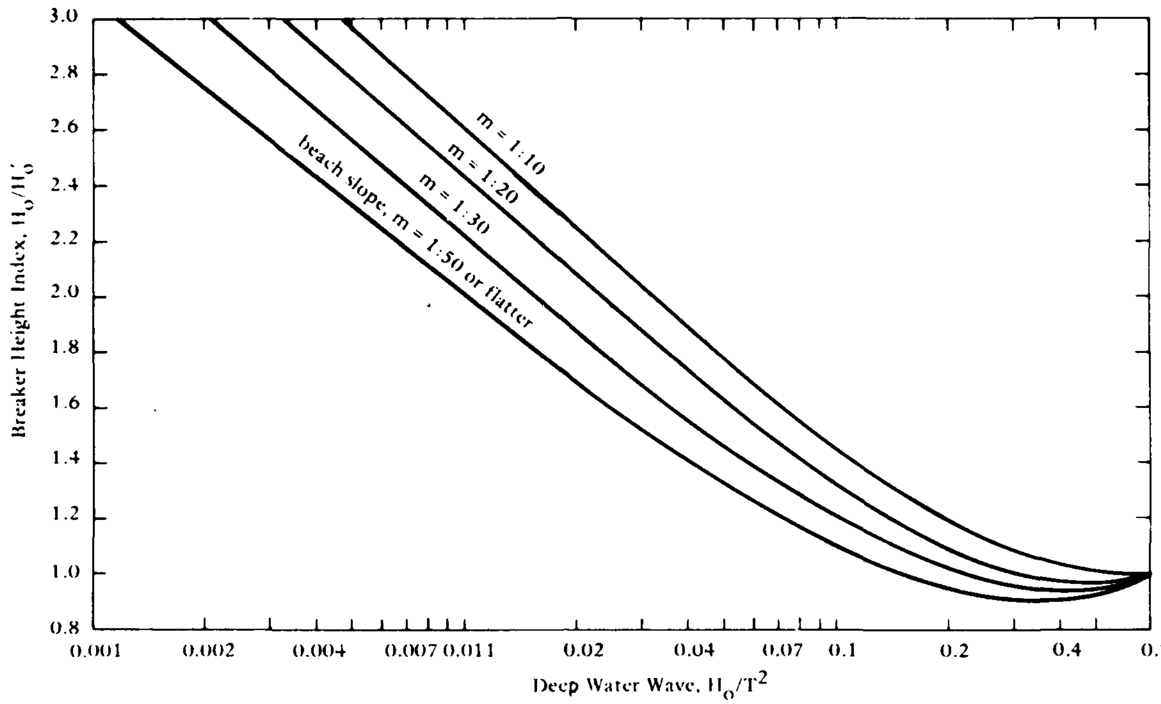


Figure 5-9. Breaker height (after Iversen, 1953).

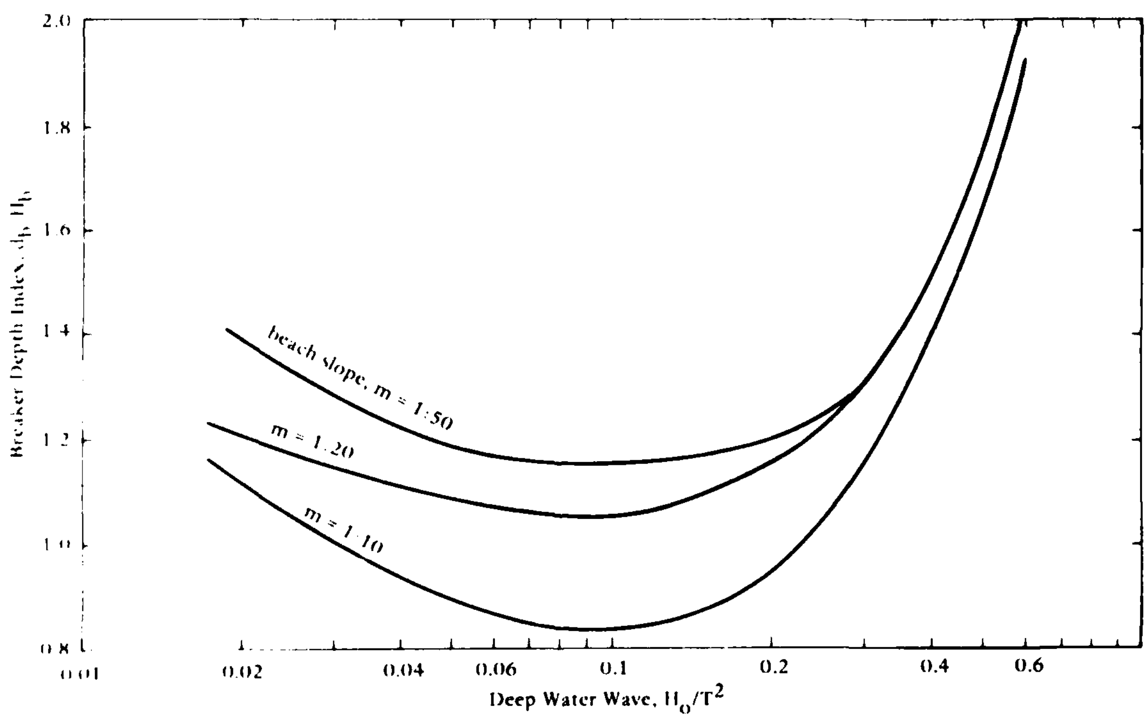


Figure 5-10. Breaking wave depth (after Iversen, 1953).

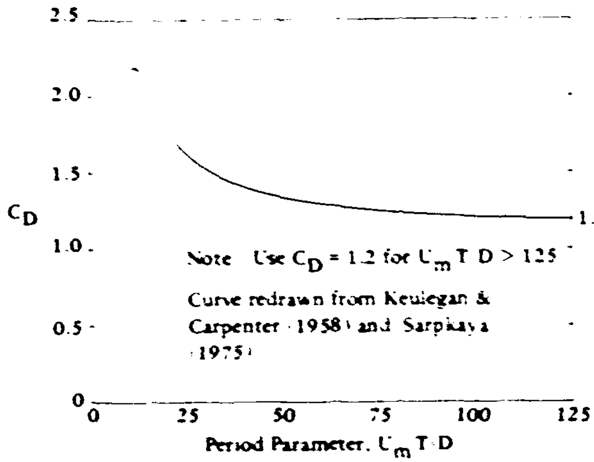


Figure 5-13. C_D versus period parameter for a smooth cylinder (from Davis and Ciani, 1976).

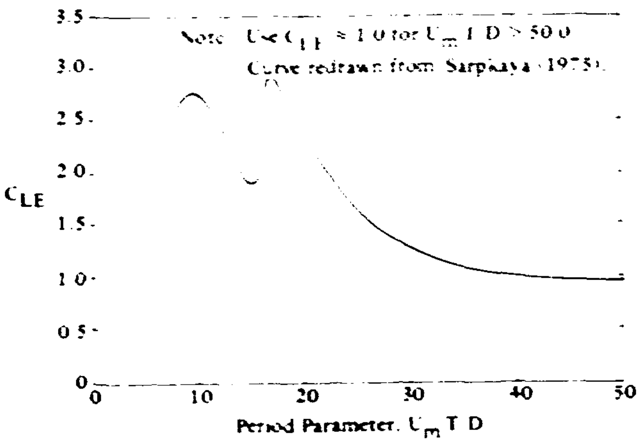


Figure 5-14 C_L versus period parameter for a smooth cylinder (from Davis and Ciani, 1976).

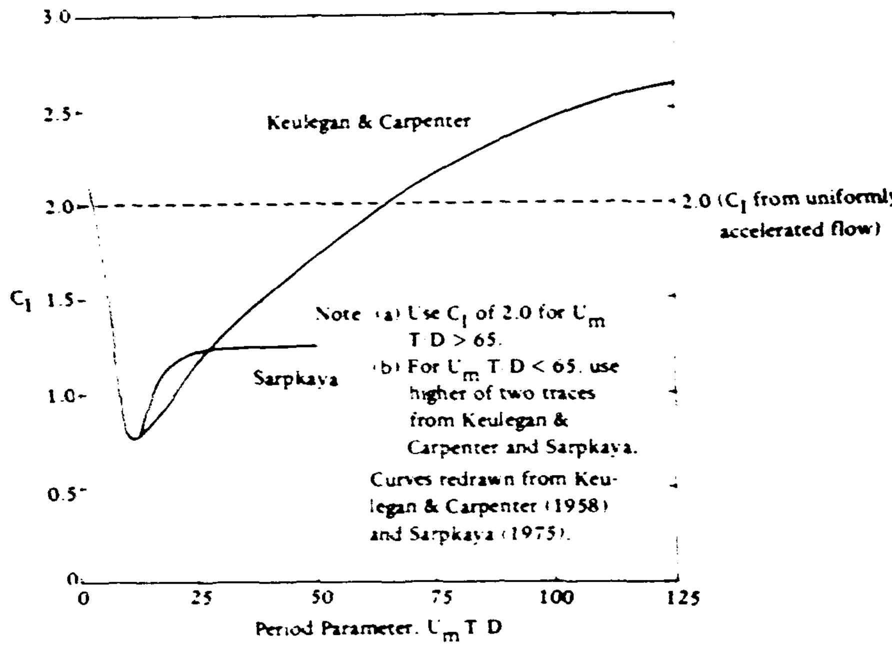
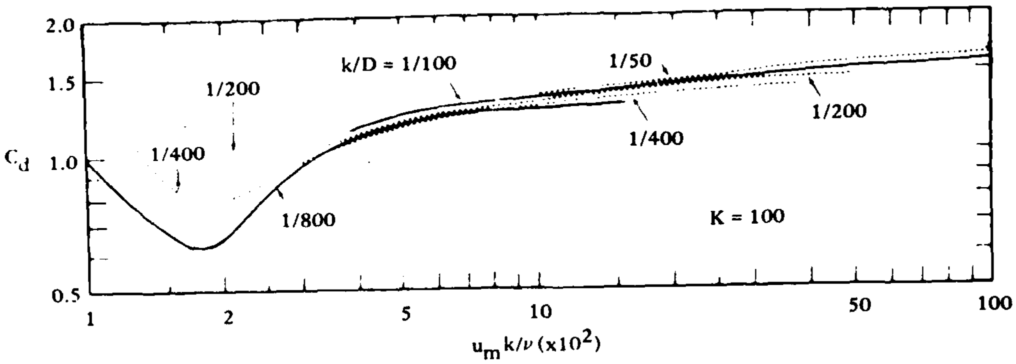
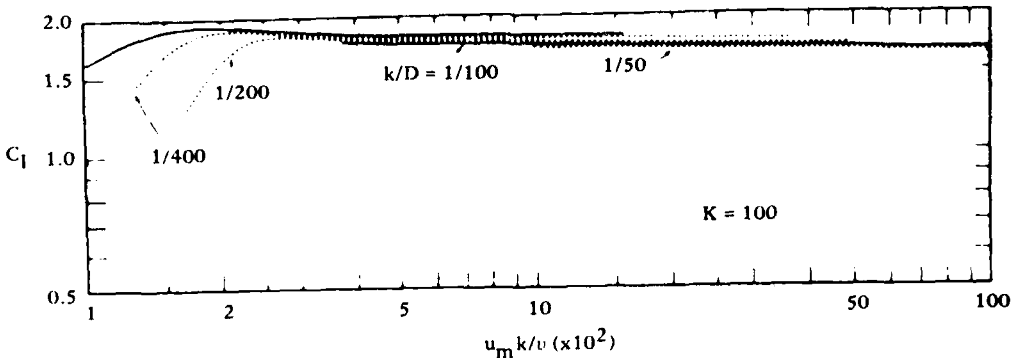


Figure 5-15. C_I versus period parameter for a smooth cylinder (from Davis and Ciani, 1976).

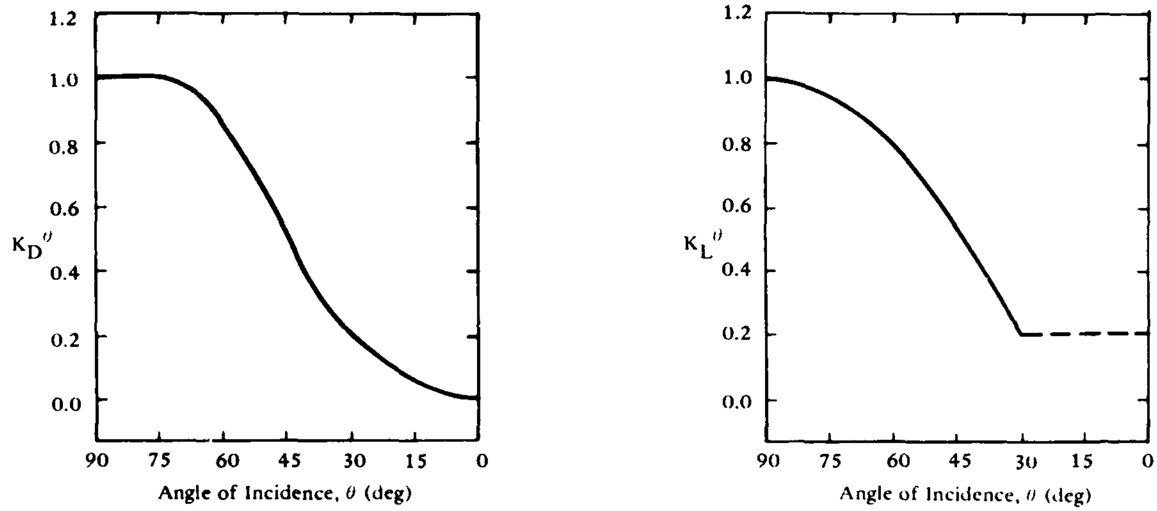


(a) Drag coefficients.



(b) Inertia coefficients.

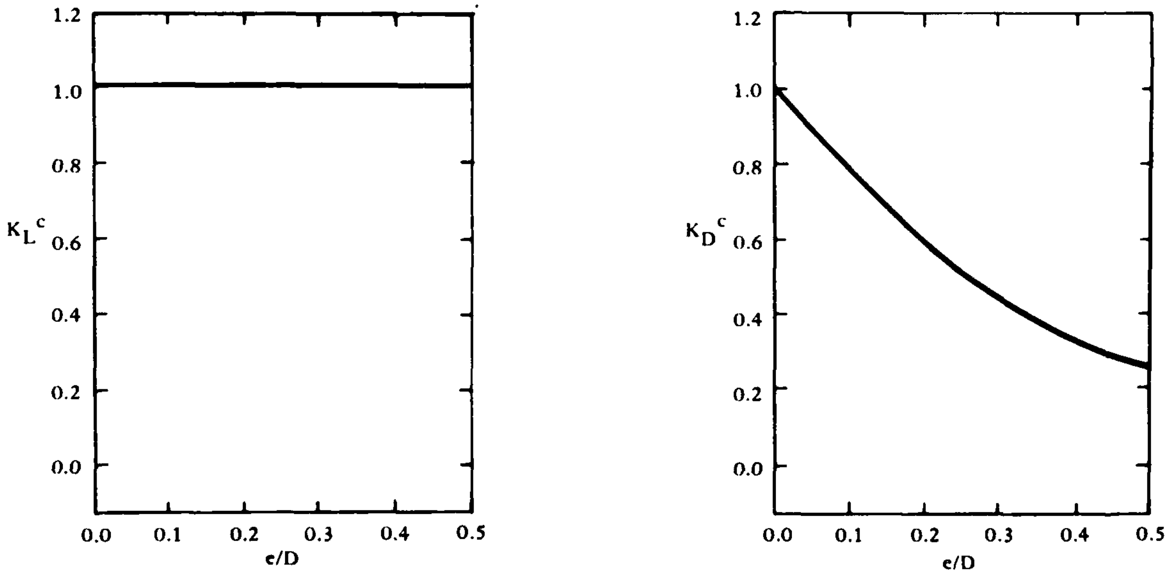
Figure 5-16. Coefficients of drag and inertia versus roughness Reynold's number for period parameter, K = 100 (from Sarpkaya, 1977).



(a) C_D curve.

(b) C_L curve.

Figure 5-17. Effect of angle of incidence of velocity vector on C_D and C_L (after Cullison, 1975).



(a) C_D curve.

(b) C_L curve.

Figure 5-18. Effect of clearance between seafloor and cable on values of C_D and C_L (after Cullison, 1975).

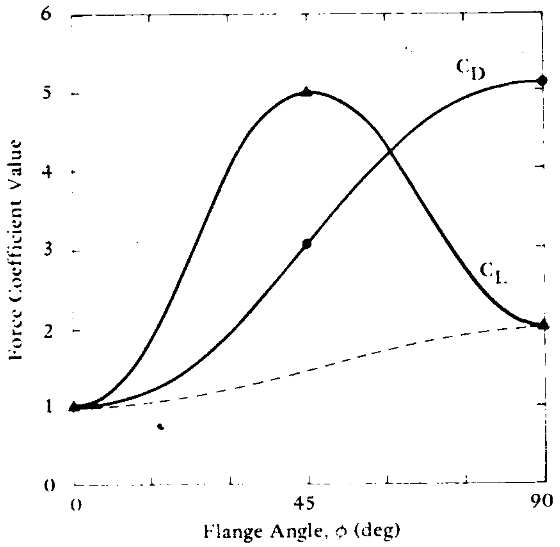


Figure 5-19. Force coefficients versus ϕ for K = 25.

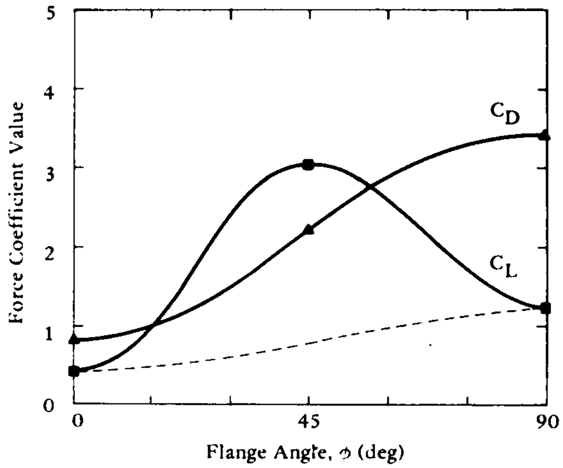


Figure 5-20. Force coefficients versus ϕ for K = 50.

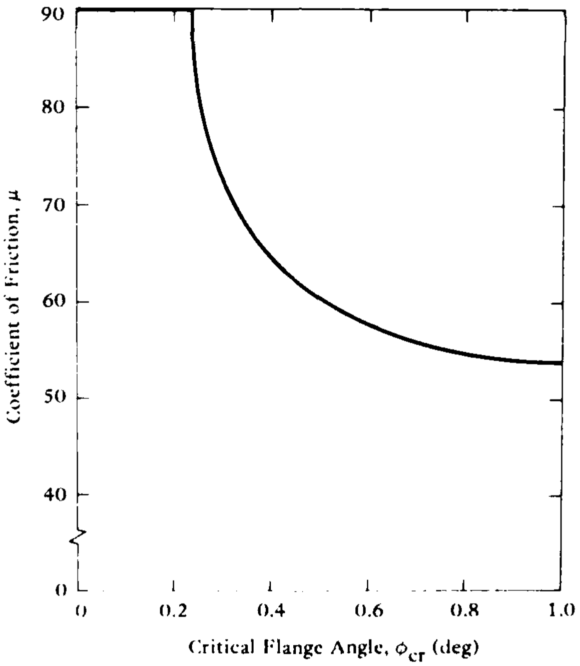


Figure 5-21. Split-pipe critical flange angle versus coefficient of friction.

Table 6-1. Effect of Clamp Configuration on Deflection and Maximum Allowable Tension of Cable Immobilization Systems

Configuration	h	Typical h/d_B	Δ_2	$T_{\Delta max}$
1 - Strap Clamp	variable	>2	$\dfrac{64(T_\Delta - \mu_s N T_B) h^3}{3 E_B \pi d_B^4}$	$\dfrac{N \pi d_B^3}{32 h} \sigma_y^B + \mu_s N T_B$
2 - Block Clamp	D	>2	$\dfrac{64(T_\Delta - \mu_s N T_B) D^3}{3 E_B \pi d_B^4}$	$\dfrac{N \pi d_B^3}{32 D} \sigma_y^B + \mu_s N T_B$
3 - Double Block Clamp	0	0	0	$\dfrac{N \pi d_B^2}{4} \sigma_y^B + \mu_s N T_B$
4 - Split-Pipe	3.5 in.	5.6	$(6.4\times10^{-5}$ in./lb$)(T_\Delta - \mu_B N T_B)$	$N(616$ lb $+ \mu_s T_B)$

Table 6-2. Mechanical Properties of Materials Most Commonly Used in Ocean Cables

Material	Modulus of Elasticity, E (psi)	Poisson's Ratio, μ_p	Yield Stress, σ_y (psi x 10^3)	Ultimate Stress, σ_u (psi x 10^3)
Steel (armor wire)	30×10^6	0.28-0.29	30-40	50-65
Steel, high strength	30×10^6	0.28-0.29	40-80	65-90
Copper	15.6×10^6	0.355	5	32
Lead				
Cast iron	$13\text{-}21 \times 10^6$	0.21-0.29	8-40	18-60
Stainless steel	27.6×10^6	0.30	30-35	85-95
Polyethylene				
High density	$0.8\text{-}1.5 \times 10^5$			3-5.5
Low density	$0.17\text{-}0.35 \times 10^5$			1-2.3

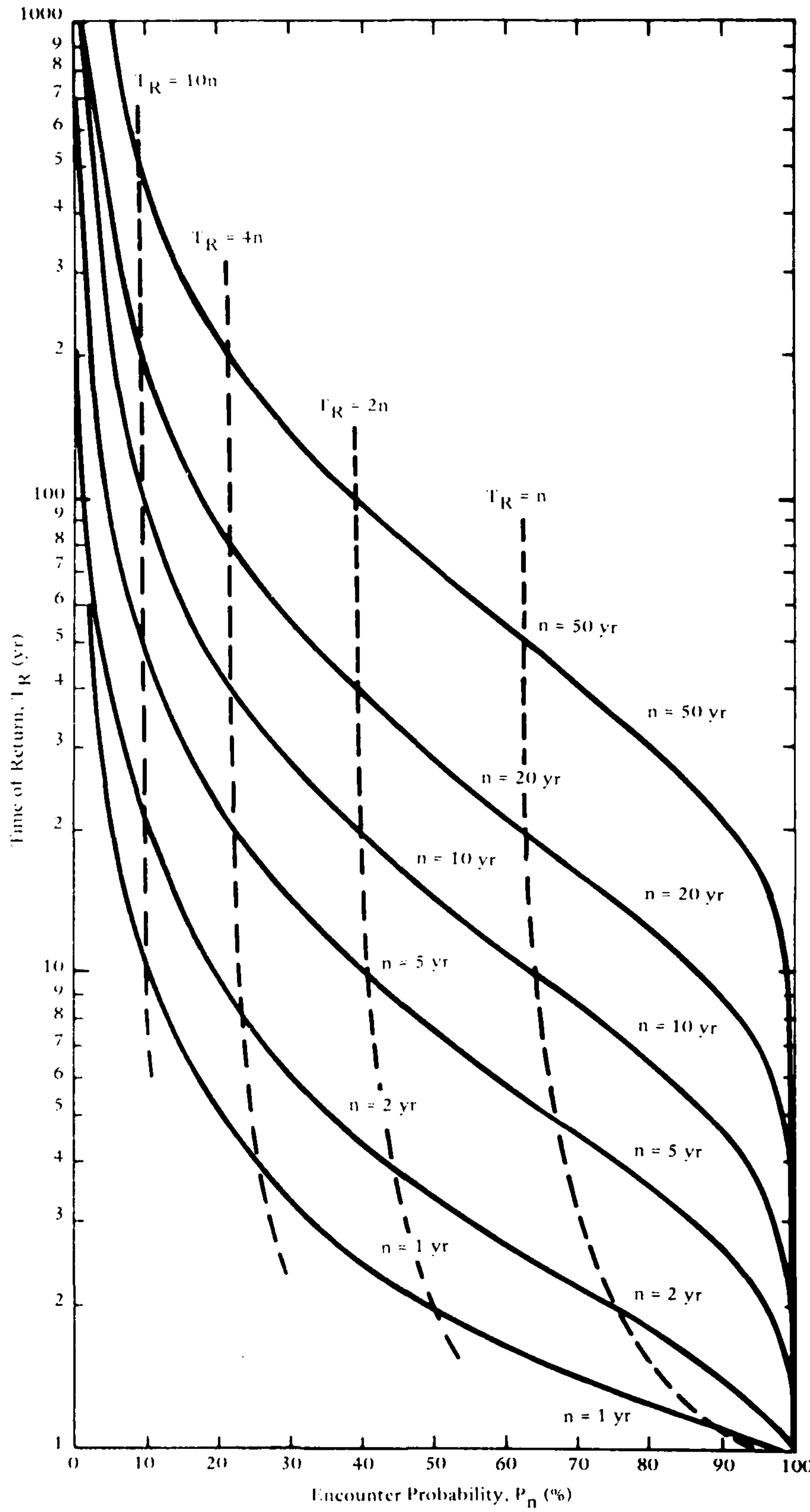


Figure 7-1. Encounter probability versus time of return for various installation life requirements.

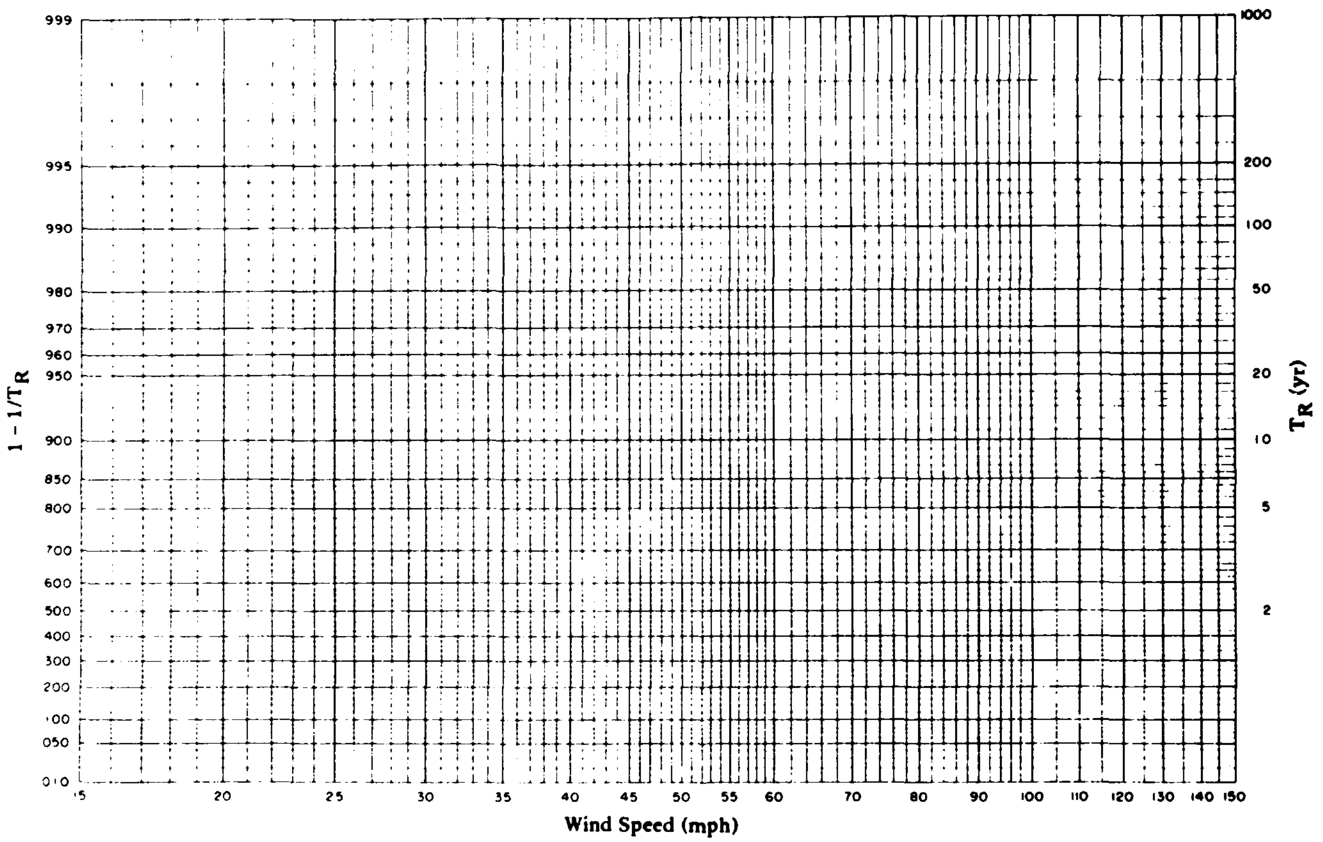


Figure 7-2. Logarithmic probability paper for maximum wind speed (Fischer-Tippett Type II distribution).

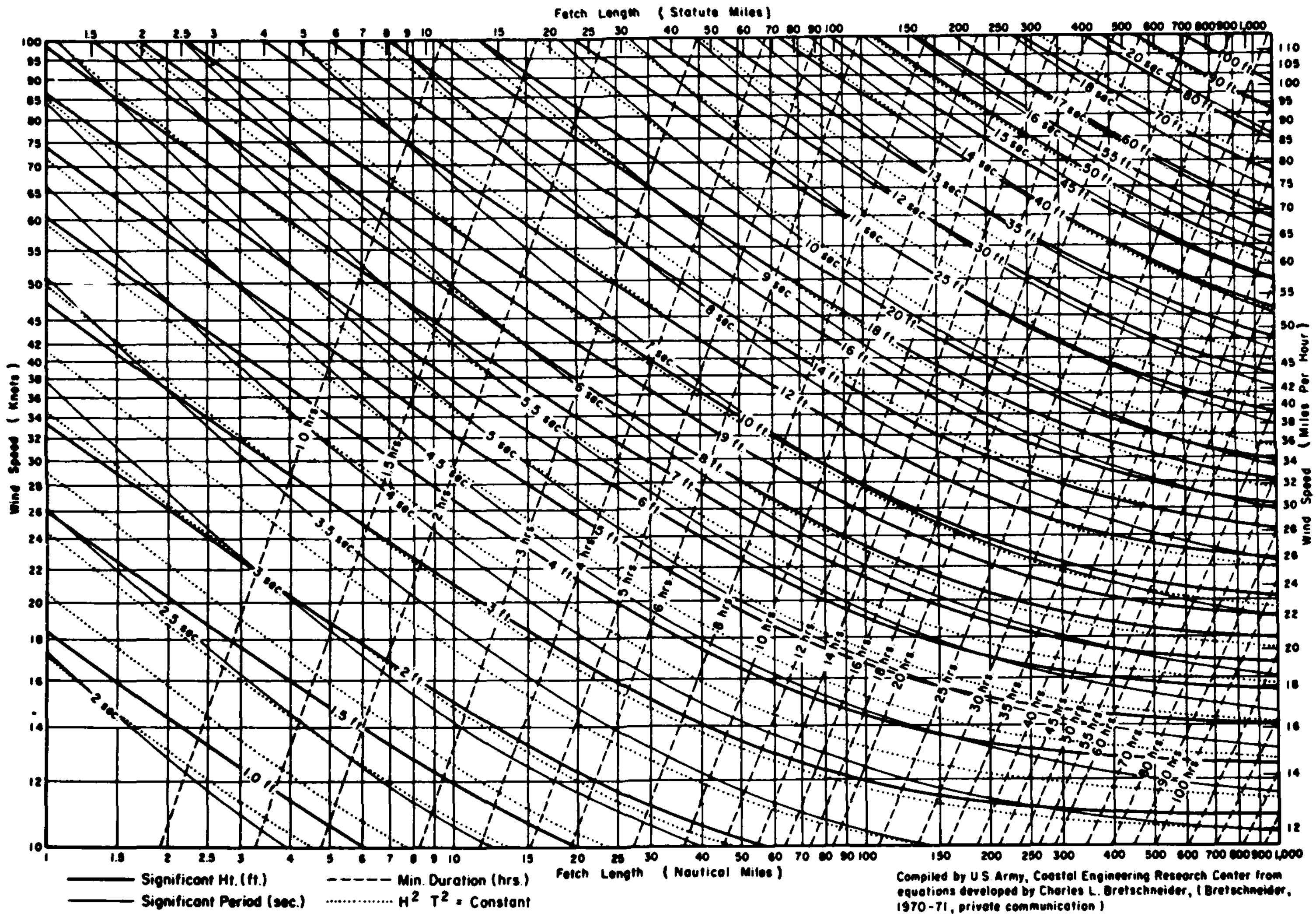


Figure 7-3. Forecasting curves for wave height and period (SMB method).

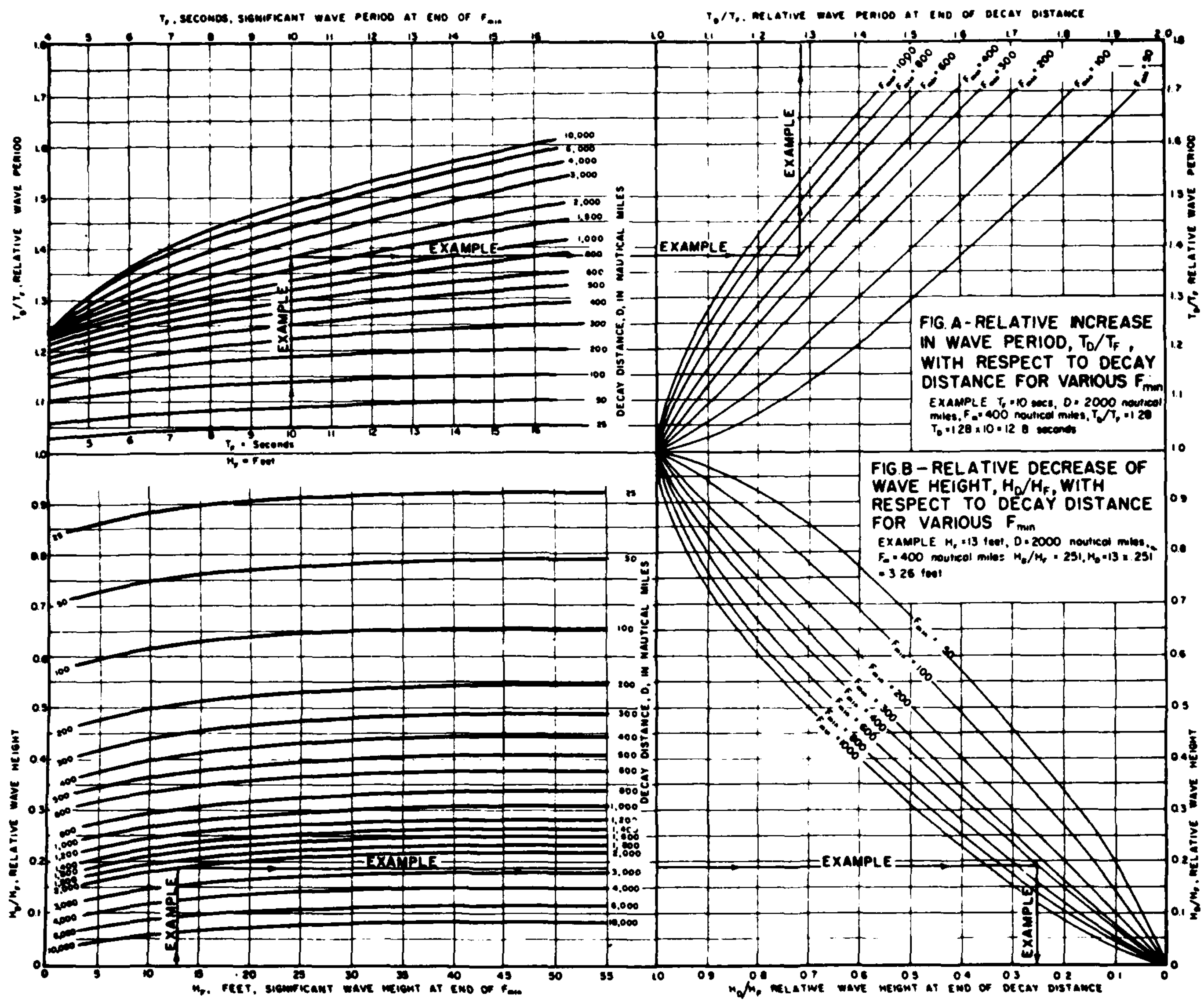


Figure 7-4. Decay curves (from Army CERC, 1973).

Appendix E

CABLE PROTECTION SYSTEM DESIGN EQUATIONS

WAVE PROFILE

$$\eta = \frac{H}{2} \cos\left(\frac{2\pi}{L} x + \frac{2\pi}{T} t\right) \qquad (2\text{-}1)$$

CABLE BREAKING STRENGTH

$$F_B = \left[\frac{\pi}{d_w}(D_c + d_w) \; \frac{\pi}{4} \sigma_u \, d_w^2\right] \qquad (4\text{-}1)$$

If $d_w = \overline{d_w}$, then:

$$F_B = \frac{\pi^2}{4} \sigma_u (D_c + d_w) d_w \qquad (4\text{-}2)$$

ROCK BOLT PRETENSION LOAD

$$T_B = \frac{2N \, T_N}{d_B \left\{ \left[\dfrac{\sin \alpha_t + \mu_o(\cos \alpha_t / \cos \beta_t)}{\cos \alpha_t - \mu_o(\sin \alpha_t / \cos \beta_t)}\right] + \mu_N \dfrac{d_N}{d_B} \right\}} \qquad (4\text{-}3)$$

where:

$$\alpha_t = \tan^{-1}\left(\frac{1}{n_t \pi d_B}\right) \quad \text{and} \quad \beta_t = 30^\circ \tag{4-4}$$

4. EXPLOSIVE BURDEN

$$B_r = 37.8 D_E \left(\frac{\rho_E}{\rho_R}\right)^{1/3} \tag{4-5}$$

5. SUBMERGED WEIGHT OF CABLES

$$W_s^* = \sum \rho_i V_i^* - \rho_w \sum V_i^* \tag{5-1}$$

6. MINIMUM CABLE TENSION DUE TO SUSPENSIONS (PARABOLIC)

$$T_o = \frac{W_s^* \ell_s^2}{2 S} \tag{5-2}$$

7. MAXIMUM CABLE TENSION DUE TO SUSPENSIONS (PARABOLIC)

$$T_{max} = W_s^* \ell \left(\frac{\ell_s^2}{4 S^2} + 1\right)^{1/2} \tag{5-3}$$

8. MINIMUM CABLE TENSION DUE TO SUSPENSIONS (CATENARY)

$$T_o = W_s^* c \tag{5-4}$$

9. MAXIMUM CABLE TENSION DUE TO SUSPENSIONS (CATENARY)

$$T_{max} = W_s^* c \cosh \frac{\ell_s}{c} \tag{5-5}$$

10. PARAMETER OF THE CATENARY

$$c = \frac{\ell_c{}^2 - S^2}{2\,S} \tag{5-6}$$

11. SEAFLOOR REACTION FORCES (PARABOLIC)

$$F_x = T_o = \frac{W_s^* \ell_s{}^2}{2\,S} \tag{5-7}$$

$$F_z = W_s^* \ell_s \tag{5-8}$$

12. SEAFLOOR REACTION FORCES (CATENARY)

$$F_x = T_o = W_s^* \left(\frac{\ell_c{}^2 - S^2}{2\,S} \right) \tag{5-9}$$

$$F_z = W_s^* \ell_c \tag{5-10}$$

13. HORIZONTAL (DRAG) FORCE

$$F_D = \frac{1}{2} C_D \rho A u^2 \tag{5-11}$$

14. VERTICAL (LIFT) FORCE

$$F_L = \frac{1}{2} C_L \rho A u^2 \tag{5-12}$$

15. INERTIA FORCE

$$F_I = C_I \rho V \frac{du}{dt} \tag{5-13}$$

16. TOTAL HORIZONTAL FORCE

$$F_H = F_I + F_D = C_I \rho V \frac{du}{dt} + \frac{1}{2} C_D \rho A u^2 \tag{5-14}$$

17. HYDRODYNAMIC FORCES PER UNIT LENGTH OF CABLE

$$F_D^* = \frac{1}{2} C_D \rho D u^2 \tag{5-15}$$

$$F_L^* = \frac{1}{2} C_L \rho D u^2 \tag{5-16}$$

$$F_I^* = \frac{\pi}{4} C_I \rho D^2 \frac{du}{dt} \tag{5-17}$$

18. SEA SURFACE (WAVE) PROFILE

$$\eta = \frac{H}{2} \cos \left(\frac{2 \pi x}{L} - \frac{2 \pi t}{T} \right) \tag{5-18}$$

19. HORIZONTAL WATER PARTICLE VELOCITY (LINEAR THEORY)

$$u = \frac{H \pi}{T} \left\{ \frac{\cosh \left[\frac{2 \pi}{L} (d + z) \right]}{\sinh \left(\frac{2 \pi d}{L} \right)} \right\} \cos \left(\frac{2 \pi x}{L} - \frac{2 \pi t}{T} \right) \tag{5-19}$$

20. VERTICAL WATER PARTICLE VELOCITY (LINEAR THEORY)

$$v = \frac{H \pi}{T} \left\{ \frac{\sinh \left[\frac{2 \pi}{L} (d + z) \right]}{\sinh \left(\frac{2 \pi d}{L} \right)} \right\} \sin \left(\frac{2 \pi x}{L} - \frac{2 \pi t}{T} \right) \tag{5-20}$$

21. MAXIMUM HORIZONTAL WATER PARTICLE VELOCITY

$$u_{max} = \frac{H \pi}{T} \left\{ \frac{\cosh \left[\frac{2 \pi}{L} (d + 2) \right]}{\sinh \left(\frac{2 \pi d}{L} \right)} \right\} \tag{5-21}$$

22. MAXIMUM VERTICAL WATER PARTICLE VELOCITY

$$v_{max} = \frac{H \pi}{T} \left\{ \frac{\sinh \left[\frac{2 \pi}{L} (d + z) \right]}{\sinh \left(\frac{2 \pi d}{L} \right)} \right\} \tag{5-22}$$

23. MAXIMUM HORIZONTAL WATER PARTICLE ACCELERATION

$$\left(\frac{\partial u}{\partial t}\right)_{max} = \dot{u}_{max} = \frac{2\pi^2 H}{T^2} \left\{ \frac{\cosh\left[\frac{2\pi}{L}(d+z)\right]}{\sinh\left(\frac{2\pi d}{L}\right)} \right\} \tag{5-23}$$

24. MAXIMUM VERTICAL WATER PARTICLE ACCELERATION

$$\left(\frac{\partial v}{\partial t}\right)_{max} = \dot{v}_{max} = \frac{2\pi^2 H}{T^2} \left\{ \frac{\sinh\left[\frac{2\pi}{L}(d+z)\right]}{\sinh\left(\frac{2\pi d}{L}\right)} \right\} \tag{5-24}$$

25. WAVE KINEMATICS EQUATIONS FOR CABLES RESTING ON OR SUSPENDED NEAR THE SEAFLOOR

$$u_{max} = \frac{\pi H}{T \sinh\left(\frac{2\pi d}{L}\right)} \tag{5-25}$$

$$\dot{u}_{max} = \frac{K_A\, 2\pi^2 H}{T^2 \sinh\left(\frac{2\pi d}{L}\right)} = \frac{3\pi}{T} u_{max} \tag{5-26}$$

$$v_{max} = \dot{v}_{max} = 0 \tag{5-27}$$

26. CHANGE IN WAVE HEIGHT DUE TO SHOALING

$$\frac{H}{H'_o} = \left\{ \tanh \frac{2\pi d}{L} \left[1 + \frac{4\pi d/L}{\sinh(4\pi d/L)}\right] \right\}^{-1/2} \tag{5-28}$$

27. CHANGE IN WAVE LENGTH DUE TO SHOALING

$$\frac{L}{L_o} = \tanh\left(\frac{2 \pi d}{L}\right) \tag{5-29}$$

28. SNELL'S LAW FOR WAVE REFRACTION

$$\frac{\sin \alpha}{\sin \alpha_o} = \frac{C}{C_o} \tag{5-30}$$

29. WAVE REFRACTION ANGLE

$$\alpha = \sin^{-1}\left(\tanh \frac{2 \pi d}{L} \sin \alpha_o\right) \tag{5-31}$$

30. CHANGE IN WAVE HEIGHT DUE TO REFRACTION

$$\frac{H_R}{H_o} = \left(\frac{b_o}{b_R}\right)^{1/2} \tag{5-32}$$

31. REFRACTION COEFFICIENT

$$K_R = \left(\frac{b_o}{b_R}\right)^{1/2} = \left(\frac{\cos \alpha_o}{\cos \alpha}\right)^{1/2} \tag{5-33}$$

32. CHANGE IN WAVE HEIGHT DUE TO SHOALING AND REFRACTION

$$H = H_o\left(\frac{H}{H'_o}\right) K_R \tag{5-34}$$

$$\frac{H}{H_o} = K_R K_s \tag{5-35}$$

33. WAVE SPEED RATIO

$$\frac{C_1}{C_2} = \frac{\tanh(2\pi d_1/L)}{\tanh(2\pi d_2/L)} \tag{5-36}$$

34. MAGNITUDE OF COMBINED WAVE AND CURRENT VELOCITY

$$u^2 = (u_w \sin \alpha + u_c \sin \beta)^2 + (u_w \cos \alpha + u_c \cos \beta)^2 \tag{5-37}$$

35. DIRECTION OF COMBINED WAVE AND CURRENT VELOCITY

$$\theta = \psi + \tan^{-1}\left(\frac{u_w \sin \alpha + u_c \sin \beta}{u_w \cos \alpha + u_c \cos \beta}\right) \tag{5-38}$$

36. REYNOLDS NUMBER

$$R_e = \frac{u\,D}{\nu} \tag{5-39}$$

37. KENLEGAN-CARPENTER PERIOD PARAMETER

$$K = \frac{U_m T}{D} \tag{5-40}$$

38. ROUGHNESS REYNOLDS NUMBER

$$R_e^K = R_e\left(\frac{k}{D}\right) = \frac{u_m\, k}{\nu} \qquad (5\text{-}41)$$

39. EFFECTIVE (COMBINED) FORCE COEFFICIENTS

$$C_D = K_D^{\theta}\, K_D^{C}\, C'_D \qquad (5\text{-}42)$$

$$C_L = K_L^{\theta}\, K_L^{C}\, C'_L \qquad (5\text{-}43)$$

$$C_I = C'_I \qquad (5\text{-}44)$$

40. CABLE STRUMMING FREQUENCY

$$f = S_n \frac{u}{D} \qquad (5\text{-}45)$$

41. CABLE STABILITY CRITERIA

$$C_D + \mu\, C_L < \frac{2\, \mu\, W_s^*}{\rho\, A\, u_{max}^2} \qquad (5\text{-}46)$$

42. SPLIT-PIPE FORCE COEFFICIENTS AS A FUNCTION OF FLANGE ANGLE

$$C_L = -A\cos(4\phi) - B\cos(2\phi) + C \qquad (5\text{-}47)$$

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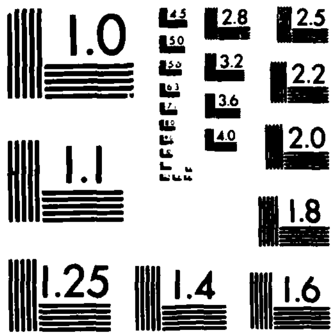

MICROCOPY RESOLUTION TEST CHART
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$$C_D = -D \cos(2\phi) + E \tag{5-48}$$

43. CRITICAL FLANGE ANGLE FOR SPLIT-PIPE

$$\phi_{cr} = \sin^{-1}\left(\frac{D + \mu B + 4 \mu A}{8 \mu A}\right)^{1/2} \tag{5-49}$$

44. SEAFLOOR REACTION FORCE PER UNIT LENGTH

$$F_N^* = \begin{cases} W_S^* - F_L^* & \text{for } F_L^* < W_S^* \\ 0 & \text{for } F_L^* > W_S^* \end{cases} \tag{6-1}$$

45. NET HORIZONTAL FORCE PER UNIT LENGTH

$$F_H^* = F_D^* + F_I^* - \mu F_N^* \tag{6-2}$$

$$F_H^* = \frac{1}{2} C_D \rho_w D u^2 + \frac{\pi}{4} C_I \rho_w D^2 \frac{du}{dt} - \mu \left(W_s^* - \frac{1}{2} C_L \rho_w D u^2\right) \tag{6-3a}$$

46. NET HORIZONTAL FORCE PER UNIT LENGTH (EXCLUDING INERTIA FORCE)

$$F_H^* = \frac{1}{2} (C_D + \mu C_L) \rho_w D u^2 - \mu W_s^* \tag{6-3b}$$

47. CABLE DEFLECTION DUE TO UNIFORMLY DISTRIBUTED HORIZONTAL FORCE

$$\delta = \frac{F_H^* \ell^2}{2(T_I + T_\Delta)} \tag{6-4}$$

48. CABLE ELONGATION DUE TO INTERNAL STRAIN

$$\Delta_1 = \frac{4 T_\Delta \ell}{\pi E_c D^2} \tag{6-5}$$

49. SEAFLOOR REACTION FORCE DUE TO PRETENSION LOAD OF IMMOBILIZATION SYSTEM

$$F_N{}^c = F_N{}^s = N T_B \tag{6-6}$$

50. IMMOBILIZATION POINT STABILITY CRITERIA

$$0 \geqq T_\Delta \geqq \mu_s N T_B \tag{6-7}$$

51. CABLE DISPLACEMENT DUE TO DEFLECTION OF IMMOBILIZATION SYSTEM

$$\Delta_2 = \left[T_\Delta - \mu_s N T_B\right]\left[\frac{h^3}{3 E_B I_B} + \frac{h}{G_B A_B}\right] \tag{6-8}$$

For immobilization fasteners with circular cross section

$$\Delta_2 = \frac{8\ T\ h}{\pi\ E_B\ d_B^{\ 2}} \left[\frac{8}{3} \left(\frac{h}{d_B} \right)^2 + (1 + \mu_p) \right] \qquad (6\text{-}9)$$

Neglecting deflection due to shear

$$\Delta_2 = \frac{(T_\Delta - \mu_s\ N\ T_B)\ h^3}{3\ E_B\ I_B} \qquad (6\text{-}10)$$

52. MAXIMUM TENSION BEFORE CABLE SLIPS THROUGH CLAMP

$$T_\Delta = (\mu_s + \mu_c)\ N\ T_B \qquad (6\text{-}11)$$

53. MAXIMUM TENSION PRIOR TO PLASTIC DEFORMATION OF THE IMMOBILIZATION FASTENER

$$T_\Delta = \mu_s\ N\ T_B + \frac{N\ \pi\ d_B^{\ 3}}{16\ M\ h}\ \sigma_y^{\ B} \qquad (6\text{-}12)$$

where $$M = 1 + \left[1 + \left(0.25\ \frac{d_B}{h} \right)^2 \right]^{1/2}$$

In the limit as $h/d_B \rightarrow \infty$:

$$T_\Delta = \frac{N\ \pi\ d_B^{\ 3}}{32\ h}\ \sigma_y^{\ B} + \mu_s\ N\ T_B \qquad (6\text{-}13)$$

In the limit as $h/d_B \rightarrow 0$:

$$T_\Delta = \frac{N \pi d_B^2}{4} \sigma_y^B + \mu_s N T_B \tag{6-14}$$

54. BINOMIAL EXPANSION APPROXIMATION OF CABLE LENGTH

$$\ell_c \approx \ell \left[1 + \frac{2}{3} \left(\frac{\delta}{\ell} \right)^2 \right] \tag{6-15}$$

55. TOTAL DEFLECTION OF CABLE MIDWAY BETWEEN IMMOBILIZATION POINTS

$$\delta = \left[\frac{6 T_\Delta \ell^2}{\pi E_c D^2} + \frac{(T_\Delta - \mu N T_B) h^3 \ell}{2 N E_B I_B} \right]^{1/2} \tag{6-16}$$

56. GENERAL DESIGN EQUATIONS FOR CABLE IMMOBILIZATION

$$\ell^3 = \psi_\delta (T_I + T_\Delta)^2 T_\Delta \tag{6-17}$$

$$\delta^2 = \psi_{T_\Delta} \ell^2 \tag{6-18}$$

$$\delta^2 = \psi_\ell (T_I + T_\Delta) T_\Delta \tag{6-19}$$

57. TRANSFER FUNCTIONS FOR GENERAL DESIGN EQUATIONS

$$\psi_{\delta} = \left[\frac{24\ \ell}{\pi\ F_H^{*2}\ E_c\ D^2} + \frac{2\ h^3 \left(1 - \frac{\mu_s\ N\ T_B}{T_\Delta}\right)}{N\ F_H^{*2}\ E_B\ I_B} \right] \qquad (6\text{-}20)$$

$$\psi_{T_\Delta} = \frac{6\left(\frac{F_H^*\ \ell^2}{2\ \delta} - T_I\right)}{\pi\ E_c\ D^2} + \frac{\left(\frac{F_H^*\ \ell^2}{2\ \delta} - T_I - \mu\ N\ T_B\right) h^3}{2\ N\ E_B\ I_B\ \ell} \qquad (6\text{-}21)$$

$$\psi_{\ell} = \left\{ \frac{12\ \delta}{\pi\ E_c\ D^2\ F_H^*} + \left[\frac{2\ \delta}{F_H^*(T_I + T_\Delta)}\right]^{1/2} \left[\frac{h^3\left(1 - \frac{\mu\ N\ T_B}{T_\Delta}\right)}{2\ N\ E_B\ I_B}\right] \right\} \qquad (6\text{-}22)$$

58. DESIGN LOAD CONDITION 1

$$\left.\begin{array}{l} T_\Delta < \mu_s\ N\ T_B \\ \text{and} \\ T_I + T_\Delta < \sigma_y{}^c\ \frac{\pi\ D^2}{4} \end{array}\right\} \qquad (6\text{-}23)$$

59. DESIGN LOAD CONDITION 2

$$\left.\begin{array}{l} T_\Delta < \sigma_y^{\,B} \dfrac{N \pi d_B^{\,3}}{32\ h} \\ \text{and} \\ T_I + T_\Delta < \sigma_y^{\,c} \dfrac{\pi D^2}{4} \\ \text{and} \\ T_B = 0 \end{array}\right\} \tag{6-24}$$

60. DESIGN LOAD CONDITION EVALUATION PARAMETER

$$\chi = \frac{\sigma_y^{\,B} \pi d_B^{\,3}}{32\ h\ \mu_s\ T_B} \left[1 + \frac{3\ h^3\ \pi^{3/2}\ F_H^* \ E_c^{\,3/2}\ D^3}{(24\ \mu\ N\ T_B)^{3/2}\ N\ E_B\ I_B}\right]^{1/3} \tag{6-25}$$

61. IMMOBILIZATION DESIGN FOR MAXIMUM ALLOWABLE LOAD AND $\chi > 1$

$$\ell = \left(\frac{24\ \hat{T}}{\pi\ F_H^{*2}\ E_c\ D^2}\right)^{1/2} \tag{6-26}$$

Case	T_I	$\hat{T}$
1	0	$(\mu_s\ N\ T_B)^3$
2	$> \sigma_y^{\,c} \dfrac{\pi D^2}{4} - \mu_s\ N\ T_B$	$\left(\dfrac{\sigma_y^{\,c}\ \pi\ D^2}{4}\right)^2 \left(\dfrac{\sigma_y^{\,c}\ \pi\ D^2}{4} - T_I\right)$

(continued)

Case	T_I	$\hat{T}$
3	$< \sigma_y^c \frac{\pi D^2}{4} - \mu_s N T_B$	$(\mu_s N T_B + T_I)^2 (\mu_s N T_B)$
4	adjustable	$\left(\frac{\sigma_y^c \pi D^2}{4}\right)^2 (\mu N T_B)$

62. OPTIMUM CABLE PRETENSION FOR MAXIMUM ALLOWABLE LOAD AND $\chi > 1$

$$T_I = \sigma_y^c \frac{\pi D^2}{4} - \mu_s N T_B \tag{6-27}$$

63. IMMOBILIZATION DESIGN FOR MAXIMUM ALLOWABLE LOAD AND $\chi < 1$

$$\ell^3 = \frac{24 \hat{T}}{\pi F_H^{*2} E_c D^2} \ell + \frac{2 \hat{T} h^3}{N F_H^{*2} E_B I_B} \tag{6-28}$$

Case	T_I	$\hat{T}$
1	0	$\left(\sigma_y^B \frac{N \pi d_B^3}{32 h}\right)^3$
2	$> \sigma_y^c \frac{\pi D^2}{4} - \sigma_y^B \frac{N \pi d_B^3}{32 h}$	$\left(\sigma_y^c \frac{\pi D^2}{4}\right)^2 \left(\sigma_y^B \frac{N \pi d_B^3}{32 h} - T_I\right)$
3	$< \sigma_y^c \frac{\pi D^2}{4} - \sigma_y^B \frac{N \pi d_B^3}{32 h}$	$\left(\sigma_y^B \frac{\pi d_B^3}{32 h} + T_I\right)^2 \left(\sigma_y^B \frac{N \pi d_B^3}{32 h}\right)$
4	adjustable	$\left(\sigma_y^c \frac{\pi D^2}{4}\right)^2 \left(\sigma_y^B \frac{N \pi d_B^3}{32 h}\right)$

64. SOLUTION OF EQUATION 6-28

$$p = \frac{8\,\hat{T}}{\pi\, F_H^{*2}\, E_c\, D^2} \tag{6-29}$$

$$q = \frac{\hat{T}\, h^3}{N\, F_H^{*2}\, E_B\, I_B} \tag{6-30}$$

$$\tau = q^2 - p^3 = \frac{\hat{T}^2\, h^6}{N^2\, F_H^{*4}\, E_B^{\ 2}\, I_B^{\ 2}} - \frac{512\,\hat{T}^3}{\pi^3\, F_H^{*6}\, E_c^{\ 3}\, D^6} \tag{6-31}$$

$$\ell = \left[q + (q^2 - p^3)^{1/2}\right]^{1/3} + \left[q - (q^2 - p^3)^{1/2}\right]^{1/3} \tag{6-32}$$

$$\ell = \left(\frac{\hat{T}\, h^3}{N\, F_H^{*2}\, E_B\, I_B} + \tau^{1/2}\right)^{1/3} + \left(\frac{\hat{T}\, h^3}{N\, F_H^{*2}\, E_B\, I_B} - \tau^{1/2}\right)^{1/3} \tag{6-32a}$$

$$\ell_1 = 2\, q^{1/3} \tag{6-33}$$

$$\ell_1 = 2\left(\frac{\hat{T}\, h^3}{N\, F_H^{*2}\, E_B\, I_B}\right)^{1/3} \tag{6-33a}$$

$$\ell_2 = \ell_3 = -\, q^{1/3} \tag{6-34}$$

$$\ell_2 = -\left(\frac{\hat{T}\, h^3}{N\, F_H^{*2}\, E_B\, I_B}\right)^{1/3} \tag{6-34a}$$

$$\ell_1 = 2\, p^{1/2} \cos(u/3) \tag{6-35}$$

$$\ell_1 = 2 \left(\frac{8\, \hat{T}}{\pi\, F_H^{*2}\, E_c\, D^2} \right)^{1/2} \cos(u/3) \tag{6-35a}$$

$$\ell_2 = 2\, p^{1/2} \cos(u/3 + 120^\circ) \tag{6-36}$$

$$\ell_2 = 2 \left(\frac{8\, \hat{T}}{\pi\, F_H^{*2}\, E_c\, D^2} \right)^{1/2} \cos(u/3 + 120^\circ) \tag{6-36a}$$

$$\ell_3 = 2\, p^{1/2} \cos(u/3 + 240^\circ) \tag{6-37}$$

$$\ell_3 = 2 \left(\frac{8\, \hat{T}}{\pi\, F_H^{*2}\, E_c\, D^2} \right)^{1/2} \cos(u/3 + 240^\circ)$$

where: $u = \cos^{-1}(q/p^{3/2})$ (6-38)

$$= \cos^{-1} \left[\left(\frac{\pi}{8}\right)^{3/2} \left(\frac{h^3\, F_H^*\, E_c^{3/2}\, D^3}{N\, T^{1/2}\, E_B\, I_B} \right) \right] \tag{6-38a}$$

65. IMMOBILIZATION DESIGN FOR MAXIMUM ALLOWABLE DEFLECTION AND $\chi > 1$

$$\ell^4 = \frac{2\, T_I\, \delta}{F_H^*} \ell^2 + \frac{\pi\, \delta^3\, E_c\, D^2}{3\, F_H^*} \tag{6-39}$$

66. SOLUTION OF EQUATION 6-39 IF INITIAL TENSION IN CABLE IS ZERO

$$\ell = \left(\frac{\pi \, \delta^3 \, E_c \, D^2}{3 \, F_H^*} \right)^{1/4} \tag{6-40}$$

67. SOLUTION OF EQUATION 6-39 IF $T_I > 0$ AND NOT ADJUSTABLE

$$\ell = \left[\frac{T_I \, \delta + \left(T_I^{\,2} \, \delta^2 + \frac{\pi}{3} F_H^* \, \delta^3 \, E_c \, D^2 \right)^{1/2}}{F_H^*} \right]^{1/2} \tag{6-41}$$

68. OPTIMUM CABLE PRETENSION FOR MAXIMUM ALLOWABLE DEFLECTION DESIGN AND $\chi > 1$

$$T_I = \sigma_y^{\,c} \frac{\pi \, D^2}{4} - \frac{\delta \, F_H^* \, E_c}{3 \, \sigma_y^{\,c}} \tag{6-42}$$

69. IMMOBILIZATION DESIGN FOR MAXIMUM ALLOWABLE DEFLECTION AND $\chi < 1$

$$\ell^4 + \left(\frac{\pi \, h^3 \, E_c \, D^2}{12 \, N \, E_B \, I_B} \right) \ell^3 - \left(\frac{2 \, T_I \, \delta}{F_H^*} \right) \ell^2$$

$$- \left(\frac{\pi \, \delta \, T_I \, h^3 \, E_c \, D^2}{6 \, F_H^* \, N \, E_B \, I_B} \right) \ell - \left(\frac{\pi \, \delta^3 \, E_c \, D^2}{3 \, F_H^*} \right) = 0 \tag{6-43}$$

70. SOLUTION OF EQUATION 6-43 FOR $T_I = 0$

$$\ell^4 + \left(\frac{\pi\ h^3\ E_c\ D^2}{12\ N\ E_B\ I_B}\right)\ell^3 - \left(\frac{\pi\ E_c\ D^2\ \delta^3}{3\ F_H^*}\right) = 0 \tag{6-44}$$

or

$$\left(\frac{3\ F_H^*}{\pi\ E_c\ D^2}\right)\ell + \left(\frac{F_H^*\ h^3}{4\ E_B\ I_B}\right) = \left(\frac{\delta}{\ell}\right)^3 \tag{6-45}$$

Let $A = \dfrac{3\ F_H^*}{\pi\ E_c\ D^2}$ and $B = \dfrac{F_H^*\ h^3}{4\ E_B\ I_B}$

71. ITERATIVE SOLUTION TECHNIQUE FOR EQUATION 6-45

$$\ell_{i+1} = \left[\frac{\left(\frac{\delta^3}{A}\right)}{\ell_i + \frac{B}{A}}\right]^{1/3} \tag{6-46}$$

72. OPTIMUM INITIAL TENSION FOR MAXIMUM ALLOWABLE DEFLECTION DESIGN AND $\chi < 1$

$$T_I = \sigma_y{}^c\ \frac{\pi\ D^2}{4} - \left[\frac{\delta^2}{\dfrac{3\ \delta\ \sigma_y{}^c}{E_c\ F_H^*} + \left(\dfrac{\delta\ \sigma_y{}^c\ \pi\ D^2\ h^6}{8\ F_H^*\ N^2\ E_B{}^2\ I_B{}^2}\right)^{1/2}}\right] \tag{6-47}$$

73. EFFECTIVE MODULUS OF ELASTICITY OF COMPOSITE CABLES

$$E_c = \frac{4 \sum_{i=1}^{n} E_i A_i}{\pi D^2} \tag{6-48}$$

74. AXIAL DEFLECTION OF CABLE ARMOR WIRE DUE TO TENSILE LOADING

$$\delta_w = P R^2 s_w \left\{ \frac{\cos^2 \alpha_w}{G I_p} + \frac{\sin^2 \alpha_w}{E I} - \frac{\sin^2 \alpha_w \cos^2 \alpha_w \left[\left(\frac{1}{G I_p} \right)^2 - \left(\frac{1}{E I} \right)^2 \right]}{\left(\frac{\sin^2 \alpha_w}{G I_p} + \frac{\cos^2 \alpha_w}{E I} \right)} \right\} \tag{6-49}$$

75. MODULUS-AREA COEFFICIENT FOR HELICALLY WOUND ARMOR WIRE

$$E_i A_i = \frac{\pi n_w E r^4 \sin \alpha_w (\gamma_p \sin^2 \alpha_w + \cos^2 \alpha_w)}{4R^2 (\gamma_p \sin^4 \alpha_w + 2 \sin^2 \alpha_w \cos^2 \alpha_w + \gamma_p \cos^4 \alpha_w)} \tag{6-50}$$

76. NET HORIZONTAL FORCE PER UNIT LENGTH ACTING ON A SUSPENDED CABLE

$$F_H^* = \frac{1}{2} C_D \rho D u^2 + \frac{\pi}{4} C_I \rho D^2 \frac{du}{dt} \tag{6-51}$$

77. SUBMERGED WEIGHT PER UNIT LENGTH OF COMPOSITE CABLE

$$W_S^* = \frac{\pi}{4}\left[\sum \rho_i (D_{o_i}^{\ 2} - D_{I_i}^{\ 2}) - D^2 \rho_w\right] \tag{6-52}$$

78. TOTAL RESULTANT FORCE PER UNIT LENGTH ACTING ON A SUSPENDED CABLE

$$F_c^* = \left(F_H^{*2} + W_S^{*2}\right)^{1/2} \tag{6-53}$$

79. ANGLE BETWEEN RESULTANT FORCE AND VERTICAL AXIS

$$\theta_1 = \tan^{-1}\left(\frac{F_H^*}{W_S^*}\right) \tag{6-54}$$

80. MAXIMUM TENSION OF A SUSPENDED CABLE UNDER THE INFLUENCE OF HYDRODYNAMIC LOADS

$$T_{max} = F_c^* \left(\frac{\ell_c^{\ 2} - S^2}{2\ S}\right) \cosh\left(\frac{2\ \ell_s\ S}{\ell_c^{\ 2} - S^2}\right) \tag{6-55}$$

81. SEAFLOOR REACTION FORCES AT THE SUPPORT POINTS OF A SUSPENDED CABLE UNDER THE INFLUENCE OF HYDRODYNAMIC LOADS

$$F_x = T_o = F_c^* \left(\frac{\ell_c^{\ 2} - S^2}{2\ S}\right) \tag{6-56}$$

$$F_{z'} = F_c^*\ \ell_s \tag{6-57}$$

82. ANGLE BETWEEN THE CABLE AND HORIZONTAL AXIS AT THE SUPPORT POINT OF A SUSPENSION

$$\theta_2 = \cos^{-1}\left(\frac{T_o}{T_{max}}\right) = \cos^{-1}\left[\cosh\left(\frac{2\,\ell_s\,S}{\ell_c^{\,2} - S^2}\right)\right]^{-1} \qquad (6\text{-}58)$$

83. CABLE TENSION FORCES AT THE SUPPORT POINT OF A SUSPENSION RESOLVED INTO ORTHOGONAL COMPONENTS

$$F_x = T_o = F_c^*\left(\frac{\ell_c^{\,2} - S^2}{2\,S}\right) = T_{max}\sin\theta_2 \qquad (6\text{-}59)$$

$$F_y = F_H^*\,\ell_s = T_{max}\sin\theta_2\sin\theta_1 \qquad (6\text{-}60)$$

$$F_z = W_s^*\,\ell_s = T_{max}\sin\theta_2\cos\theta_1 \qquad (6\text{-}61)$$

84. CABLE TENSION AT THE SUPPORT POINT OF A SUSPENSION

$$T' = \frac{T_{max}}{e^{\mu_s\theta_2}} \qquad (6\text{-}62)$$

85. SEAFLOOR REACTION FORCE AT THE SUPPORT POINT OF A SUSPENSION

$$F_N = \frac{F_c^*\left(\frac{\ell_c^{\,2} - S^2}{2\,S}\right)}{\left[\cos\left(\frac{\theta_2}{2}\right) + \mu\sin\left(\frac{\theta_2}{2}\right)\right]} \qquad (6\text{-}63)$$

86. FRICTION FORCE BETWEEN THE CABLE AND SEAFLOOR AT THE SUPPORT POINT OF A SUSPENSION

$$F_f = \mu F_N \tag{6-64}$$

87. REDUCTION IN CABLE TENSION AS A RESULT OF FRICTION

$$\Delta T = \mu_s W_s^* \ell' \tag{6-65}$$

88. DISTANCE FROM SUPPORT POINT AT WHICH THE INTERNAL TENSION INDUCED BY THE SUSPENSION IS REDUCED TO ZERO

$$\ell'_{cr} = \frac{F_c^* \left(\dfrac{\ell_c{}^2 - S^2}{2\,S} \right) \cosh \left(\dfrac{2\,\ell_S\,S}{\ell_c{}^2 - S^2} \right)}{\mu_s W_s^* e^{\mu_s \theta_2}} \tag{6-66}$$

89. HORIZONTAL FORCE APPLIED TO AN IMMOBILIZATION FASTENER AT THE SUPPORT POINT OF A SUSPENSION

$$F_B = \left(T'^2 + F_y{}^2 \right)^{1/2}$$

$$= \left\{ \left[\frac{F_c^* \left(\dfrac{\ell_c{}^2 - S^2}{2\,S} \right) \cosh \left(\dfrac{2\,\ell_s\,S}{\ell_c{}^2 - S^2} \right)}{e^{\mu_s \theta_2}} \right]^2 + F_H^{*2}\,\ell_s{}^2 \right\}^{1/2} \tag{6-67}$$

90. TENSION IN A BOTTOM-RESTING CABLE AS A RESULT OF A SUSPENDED SECTION OF CABLE

$$T_I = \frac{T}{e^{\mu_s \theta_2}} - \mu_s W_s^* \ell' \qquad (6\text{-}68)$$

91. ENCOUNTER PROBABILITY FOR STORM WAVE ENVIRONMENT

$$P_n = 1 - \left(1 - \frac{1}{T_R}\right)^n \qquad (7\text{-}1)$$

92. TIME OF RETURN OF A STORM WAVE ENVIRONMENT AS A FUNCTION OF ENCOUNTER PROBABILITY AND SYSTEM LIFE REQUIREMENTS

$$T_R = \frac{1}{1 - (1 - P_n)^{1/n}} \qquad (7\text{-}2)$$

93. MAXIMUM WIND VELOCITY HAVING A RETURN PERIOD OF T_R YEARS

$$\left(\frac{U}{\beta}\right)^{-\gamma} = \ln\left(\frac{T_R}{T_R - 1}\right) \qquad (7\text{-}3)$$

94. DEPTH AT WHICH ARMORED CABLE BECOMES UNSTABLE DUE TO HYDRODYNAMIC LOADS

$$\sinh\left(\frac{2\pi d}{L}\right) \geqq \sin\theta\left[\frac{\rho\, D(C_D + \mu\, C_L)}{2\,\mu_s\, W_s^*}\right]^{1/2} \qquad (7\text{-}4)$$

and

$$d = \frac{g\,T^2}{2\,\pi}\left(\frac{d}{L_o}\right) \tag{7-5}$$

95. INERTIA FORCE EVALUATION RATIO

$$\frac{F_I}{F_H} = \frac{3\,\pi\,D}{2\,H}\sinh\left(\frac{2\,\pi\,d}{L}\right) \tag{7-6}$$

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LIST OF SYMBOLS

A	Area
A_B	Cross section area of immobilization bolt or fastener
A_i	Cross section area of i^{th} circumferential component of a cable
B	Buoyant force
B_r	Burden (distance between borehole and nearest free face at instant of initiation)
b	Width of immobilization clamp
b_o	Crest length of deep water wave
b_R	Crest length of refracted wave
C	Wave speed
C_D	Drag coefficient
C_I	Inertia coefficient
C_L	Lift coefficient
C_o	Speed of deep water wave
c	Parameter of a catenary
D	Diameter of cable

D_c	Diameter of core of cable
D_E	Diameter of explosive charge
D_{I_i}	Inside diameter of i^{th} circumferential component of cable
D_{o_i}	Outside diameter of i^{th} circumferential component of cable
d	Water depth
d_b	Water depth at which wave will break
d_B	Diameter of immobilization bolt or fastener
d_N	Mean diameter of washer face
d_w	Diameter of cable armor wire including jacketing
d_w	Diameter of unjacketed cable armor wire
E	Modulus of elasticity
E_B	Modulus of elasticity of immobilization bolt or fastener
E_c	Effective modulus of elasticity of cable
E_i	Modulus of elasticity of the i^{th} circumferential component of a cable
e	Distance between bottom of cable and seafloor
F_B	Breaking strength of cable

F_c Equivalent single force acting on cable

F_D Horizontal (drag) force

F_f Friction force

F_H Total horizontal force

F_I Inertia force

F_L Vertical (lift) force

F_N Seafloor reaction force

F_N^c Normal force between immobilization clamp and cable

F_N^s Normal force between cable and seafloor due to immobilization clamp

F_x Force acting in direction of x coordinate

F_y Force acting in direction of y coordinate

F_z Force acting in direction of z coordinate

f Frequency

G Shear modulus

G_B Shear modulus of immobilization bolt or fastener

g Acceleration due to gravity

H	Wave height
H_b	Wave height at the point of breaking
H_o	Height of deep water wave
H'_o	Deep water wave height (unaffected by refraction)
H_R	Height of refracted wave
H_s	Significant wave height
H_1	Average of highest 1% of all waves
H_{10}	Average of highest 10% of all waves
$H_{1/3}$	Average of highest 1/3 of all waves
h	Distance between the seafloor and the bottom of the immobilization clamp
I	Moment of inertia of cross section
I_B	Moment of inertia of cross section of immobilization bolt or fastener
I_p	Polar moment of inertia of cross section
K	Keulegan-Carpenter period parameter
K_A	Horizontal fluid acceleration correction factor (= 1.5)
K_D^C	Drag coefficient correction factor for clearance between cable and the seafloor

K_D^{θ} Drag coefficient correction factor for flow not perpendicular to cable

K_L^{C} Lift coefficient correction factor for clearance between cable and the seafloor

K_L^{θ} Lift coefficient correction factor for flow not perpendicular to cable

K_R Refraction coefficient

K_S Shoaling coefficient

k Roughness height

L Wave length

L_c Length of cable between immobilization points

L_o Deep water wave length

ℓ Half span length of cable between immobilization points (= $L_c/2$)

ℓ_a Length of armor lay

ℓ_c Length of cable from low point of suspension to support point

ℓ_{cr}' Distance from edge of suspension where internal cable tension becomes zero

ℓ_s Horizontal distance from low point of cable suspension to support point

ℓ_w Length of spring

M	Slope of the seafloor
N	Number of immobilization fasteners per clamp
n	Number of years that cable system must remain operational
n_t	Number of bolt threads per inch
n_w	Number of armor wires per layer
P	Axial load applied to cable
P_n	Probability that the design wave parameters will be exceeded during the life of a cable insulation
R	Radius of spring (for cables: distance from center of cable to center of armor wire layer)
R_e	Reynolds number
R_e^K	Roughness Reynolds number
r	Radius of individual armor wire
S	Maximum cable sag
S_c	Compressive strength
S_h	Drill hole spacing
S_n	Strouhal number
s_w	Length of spring wire

T	Wave period
T_B	Pretension load on bolt or immobilization fastener
T_I	Initial tension in cable
T_N	Pretensioning torque applied to rock bolt
T_o	Minimum tension of a suspended cable (at low point)
T_R	Average time of return of a storm wave environment (years)
T_Δ	Tension in cable due to strain
t	Time
t_c	Thickness of immobilization clamp
U	Maximum wind speed
U_t	Traverse speed
u	Free stream velocity of a fluid in the horizontal direction
u	Acceleration of fluid in the horizontal direction
u_c	Magnitude of current velocity
u_m	Maximum fluid velocity
u_n	Velocity of fluid perpendicular to cable path
u_t	Velocity of fluid parallel to cable path

u_w	Magnitude of wave velocity
V	Total volume
V_i	Volume of the i^{th} circumferential component of cable
v	Vertical component of water particle velocity
v	Acceleration of fluid in the vertical direction
W	Weight in air
W_s	Submerged weight
x	Distance from coordinate axis
α	Angle between wave front and seafloor contour
α_o	Angle between deep water wave front and seafloor contour
α_t	Lead angle of bolt threads
α_w	Helical angle of cable armor wire
β	Windspeed probability distribution parameter
β_t	One-half included thread angle
γ	Wind speed probability distribution parameter
γ_p	$1 + \mu_p$
Δ_1	Elongation in cable due to internal strain

Δ_2	Deflection of cable and immobilization system due to bending of the seafloor fastener
δ	Deflection of cable at midspan (between immobilization points)
δ_w	Deflection of spring wire
η	Wave shape parameter
θ	Angle in degrees
μ	Coefficient of friction
μ_c	Coefficient of friction between clamp and cable
μ_N	Coefficient of friction between nut and washer
μ_o	Coefficient of thread friction
μ_p	Poisson's ratio
μ_s	Coefficient of friction between cable and seafloor
ν	Kinematic viscosity
ρ	Density
ρ_E	Density of explosive
ρ_i	Density of i^{th} circumferential component of the cable
ρ_R	Density of rock

ρ_W	Density of seawater
σ_y	Yield stress
σ_u	Ultimate tensile stress
σ_y^B	Yield stress of immobilization bolt or fastener
σ_y^c	Yield stress of cable
ϕ	Angle of split pipe flange with respect to horizontal axis
ϕ_{cr}	Critical flange angle
χ	Design condition evaluation parameter
ψ	Immobilization design equation transfer function
*	Asterisk indicates "parameter per unit length"

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